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MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

THE COMPENSATED ARCH DAM

By A. V. KARPOV[1], M. Am. Soc. C. E.

Synopsis

The paper is a theoretical analysis of an arch dam that is intended to simplify present-day design methods and bring to light the proper relationship between the profile of the dam site and the shape of the arch dam.

Conditions are investigated under which the dam can be relieved of unnecessary stresses. The compensated dam that is introduced, carries the stresses due to dead weight and water load, but it is relieved from stresses due to contraction or expansion of concrete and deflections of the foundations and abutments.

This investigation indicates a design that is believed to result in a safer and more economical structure.

I.—Methods of Designing Arch Dams

The theory of arch dam design has been given considerable attention in recent years. Quite a number of valuable papers have been published dealing with this complex problem. However, sufficient attention was not paid to the shape of the dam, which, as in the case of most structures subjected to hydrostatic loads, is an important factor that influences the safe and economical design. A definite relationship exists between the shape of the dam site and the shapes of the vertical and horizontal cross-sections of the arch dam, which can be developed only if proper consideration is given to both the gravity and arch action and to the elastic properties of the ground on which the dam rests.

In general, all arch dam designs in the past can be divided into two classes: Those in which the gravity action is neglected, and those in which it is considered. The designs in which the gravity action is neglected are based on the assumption that the entire water load is carried by the horizontal arches and the action of the vertical elements, usually called canti-

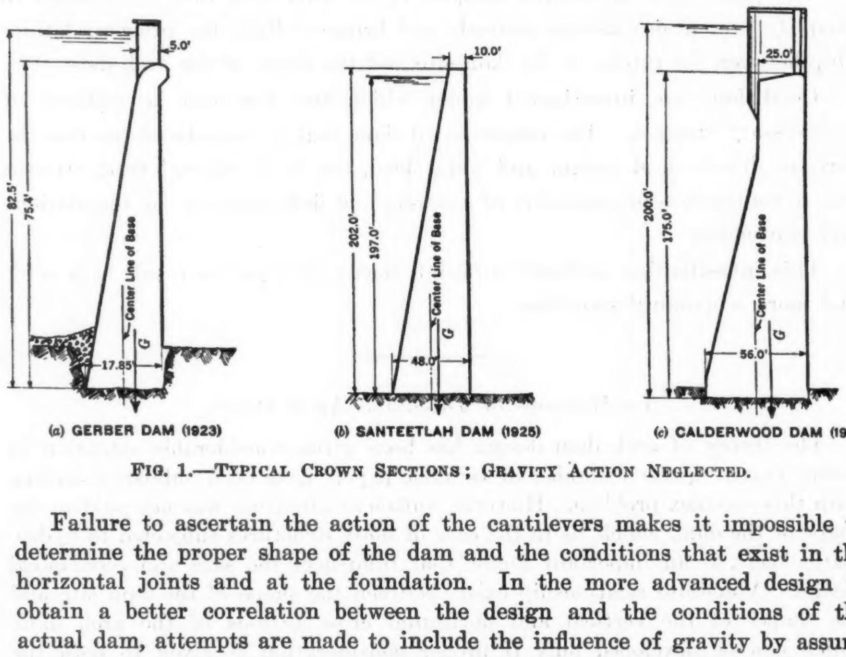
Note.—Written discussion on this paper will be closed in August, 1932, *Proceedings*.

[1] Designing Engr., Hydr. Dept., Aluminum Co. of America, Pittsburgh, Pa.

levers, is neglected under the supposition that the influence of the vertical elements is not very pronounced and merely serves to relieve the stresses in the horizontal elements.

Actually, in the lower parts of the dam, the action of the vertical elements changes the distribution of loading on the horizontal elements and may produce excessive moments that will result in heavy concentrations of stresses in some parts of the dam and uneconomically low stresses in other parts. In the higher parts, the influence of the vertical elements, or cantilevers, will be felt not only in the change of the distribution of loading, but also in the increase of the loading of the horizontal elements.

The statement is sometimes made that the influence of the vertical element may be neglected because it is counteracted by the effect of swelling when the dry up-stream face of the dam comes in contact with water. This contention seems to contradict the results of tests made on existing dams.



(a) GERBER DAM (1923) (b) SANTEETLAH DAM (1926) (c) CALDERWOOD DAM (1929)

FIG. 1.—TYPICAL CROWN SECTIONS; GRAVITY ACTION NEGLECTED.

Failure to ascertain the action of the cantilevers makes it impossible to determine the proper shape of the dam and the conditions that exist in the horizontal joints and at the foundation. In the more advanced design to obtain a better correlation between the design and the conditions of the actual dam, attempts are made to include the influence of gravity by assuming that only a part of the water load is resisted by the horizontal arches and the remainder by the vertical elements.

A difference that is most pronounced at the middle or crown can be noticed in the shapes of the vertical cross-sections. If the gravity action is neglected, the advantage of having a large central angle will be the governing factor, the larger angles showing more favorable computed stresses. The way to obtain such large angles, particularly at lower elevations, is to increase the thickness of the dam in the up-stream direction, as shown in Fig. 1, and it is customary at the same time to make the down-stream face nearly vertical.

If gravity action is considered, the central angle is not the major governing factor and vertical crown sections as shown in Fig. 2 will result, in which the up-stream face of the vertical crown element is made approximately vertical and the increased thickness is obtained by extending the lower parts of the dam in the down-stream direction.

The difference between these two types of sections is that in Fig. 1, the gravity resultant of the section will intersect the base between its middle point and its down-stream face, and, in Fig. 2, the intersection point will be between the middle point of the base and its up-stream face.

Considering the dam under water load, this difference is of importance since in the first type the support that the arch receives from the gravity action is very slight. At the same time, the appreciable bending moments that are developed in the vertical element can be taken care of only by the opening of the horizontal joints, and by a considerable increase of the maxi-

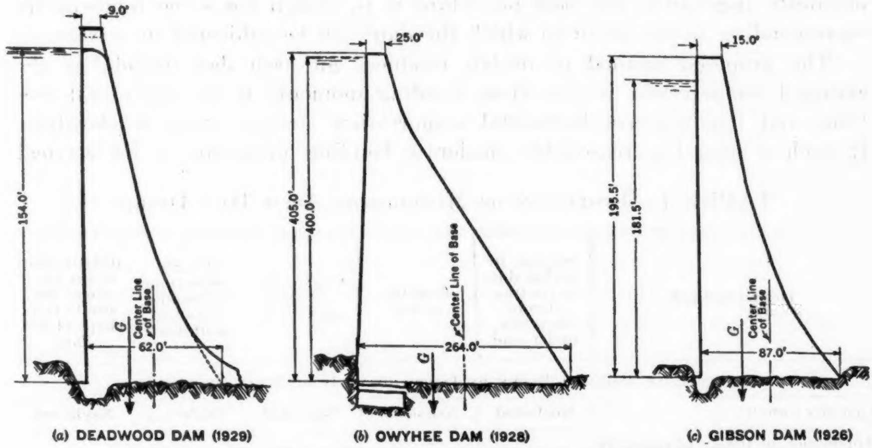


FIG. 2.—TYPICAL CROWN SECTIONS; GRAVITY ACTION CONSIDERED.

mum stresses in horizontal joints, at the foundations, and at the upper horizontal arches.

In Fig. 2 the arch not only gets all possible support from the gravity action of the vertical elements, but the bending moments in these elements are reduced, thus minimizing the opening of the horizontal joints and decreasing the stress in these joints, in the foundations, and in the upper horizontal arches. For these reasons, if for no other, the advantage of the second type (Fig. 2) is very great.

In methods used in the past the arch action of the horizontal elements of the dam was determined on the assumption of a continuous arch without joints. Actually, every commercial arch dam is built of a number of blocks with pronounced vertical, and less pronounced horizontal, joints. The design assumptions can be applied properly within the boundaries of a block, but not at the joints, where certain reservations are necessary. The most important of these is that only a limited amount of tension can be carried through

the joints, and, consequently, if the estimated stresses show tension of any magnitude, a discrepancy will exist between the computed and actual stress distribution, which can be so large as to make the computed stresses useless.

In an arch dam the number of considerations that influence the behavior of the structure and its stresses will vary, depending on the distribution of moments and stresses. In a dam that has non-uniform stresses and moments in the horizontal arches, shear and twist will be introduced, and they must be considered. If the design could be made so that the stresses are kept uniform and the moments, as well as the complications due to shear and twist, are eliminated in the horizontal arches, then a scientifically safe design influenced only by a limited number of major factors can be developed, which will result in the most economical structure. If such a design is attempted, it will be found that, for each temperature change in the concrete, some modification will be necessary in the shape of the dam. Since that is obviously impossible, the best procedure is to design for some temperature corresponding to the mean to which the dam will be subjected in service.

The proposed method of design produces an arch dam which, at the assumed temperature, is free from bending moments in its horizontal sections and has uniform horizontal compression stresses at each elevation. If such a dam has reasonably moderate bending moments in its vertical

TABLE 1.—EVOLUTION OF METHODS OF ARCH DAM DESIGN

Design method	Stresses in arches due to contraction of concrete under load	Gravity action	Continuity of the structure	Elastic deflections of foundations and abutments	Relationship of the profile of the site to this shape of the dam
ALL LOAD ASSUMED TAKEN BY INDEPENDENT HORIZONTAL ARCHES					
Cylinder formula	Neglected	Neglected	Neglected	Neglected	Neglected
Application of theory of elasticity to arches. (Cain's formula, constant-angle dam, straight voussoir method, modified voussoir methods)	Considered	Neglected	Neglected	Neglected	Neglected
LOAD DISTRIBUTED BETWEEN HORIZONTAL ARCHES AND VERTICAL ELEMENTS					
Original trial-load method (Bureau of Reclamation)	Considered	Considered	Neglected	Neglected	Neglected
Present trial-load method (Bureau of Reclamation)	Considered	Considered	Considered	Partly considered	Neglected
Compensated dam	Considered	Considered	Considered	Considered	Considered

sections, then in the vertical direction at each horizontal element there will be varying compression stresses.

The major factors that affect the behavior of such a dam and the magnitude of its stresses, at the assumed temperature, are the direct water pressure on the up-stream face, the compression in the horizontal arches, the varying vertical stresses, the horizontal shear in the vertical elements, the foundation conditions, and the conditions necessary to retain the continuity of the struc-

ture. Secondary factors that could be considered in such design are the influence of Poisson's ratio and possibly the swelling effect of the wetted concrete.

At temperatures other than the one for which the design is made bending moments will be introduced in the horizontal arches. These moments will make the horizontal stresses in the arches non-uniform and will introduce shear and twist, and the dam will be influenced by all the factors encountered in customary design. The important difference, however, is that in the customary design all these additional factors are present at any temperature and are of major importance. In the proposed design their influence on the arch will be of a secondary nature and they will occur only at temperatures differing from that assumed in the design. The interesting evolution of the different arch dam design methods is briefly summarized in Table 1.

II.—Outline of the Analysis

The simplification of the arch dam design by removing all non-essential factors is regarded as a most important step in introducing a scientific design method. The purpose of this analysis is to develop such a method by relieving the dam from bending moments in its horizontal arches. Arch dams so designed are herein designated "Compensated Arch Dams."

The procedure used for analyzing the conditions of an arch dam and for determining the main principles involved in the design is, as follows:

1.—The dam is divided into vertical elements defined by radial vertical planes, and horizontal elements defined by horizontal planes.

2.—The hydrostatic pressure applied to the dam is represented by radial forces acting at the up-stream face.

3.—A division of these forces is assumed so that they will exert varying loads on the vertical and horizontal dam elements.

4.—At first the horizontal arches are considered as a number of independent arches, each being in frictionless contact with the arch immediately above and below the one under consideration. Such an arch is free from the influence of gravity, and is restrained in the vertical direction, but has no restraint in the horizontal direction, except as provided by the abutments.

5.—In the investigation of the independent horizontal arches resultant forces and moments are taken with reference to the center line of the arch, and the proof is developed that such an arch cannot be relieved from bending moments if the assumptions are limited to changes in the shape. The problem is divided into two parts: First, to shape the horizontal arch so as to relieve it from bending moments due to external loadings; and second, to provide some means of compensation that will relieve the properly shaped arch from moments due to rib-shortening, displacements of the abutments, and difference between the assumed temperature under service and the temperature at the time of closing the dam.

6.—The shape of an arch relieved from bending moments and the necessary compensation are determined, as well as the relationship between the radial, tangential, and torsional displacements of the loaded arch for any assumed deflections of the abutments.

7.—As in the case of the horizontal elements, the vertical elements are first considered as independent, as unrestrained from the adjoining elements, and as resting with sufficient friction at the foundation and horizontal joints to prevent slipping at any elevation. The difference in assumptions with regard to the horizontal arches and vertical elements relieves the horizontal arches from the influence of gravity and concentrates it on the vertical elements. The radial, tangential, and torsional displacements of the vertical element, due to the part of the water load taken care of by the element, are analyzed.

8.—The conditions imposed upon the independent horizontal arches and the vertical elements by the necessity of retaining the continuity of the structure are investigated. Continuity manifests itself in the form of a mutual restraint between the adjoining horizontal arches and the vertical elements, and requires the equalization of the radial, tangential, and torsional deflections of the independent horizontal arches and vertical elements. The conditions are formulated under which this requirement can be satisfied and by which the independent compensated horizontal arches can be transformed into elements that are restrained in the horizontal as well as in the vertical directions, at the same time remaining compensated and relieved from any bending moments. In this way the original assumption (independent frictionless horizontal arches and independent vertical elements) is eliminated and the structure is transformed into one that corresponds to the dam as actually built.

9.—Theoretical investigations are extended to cover unsymmetrical as well as symmetrical dam sites, and the influence of temperature changes, expansion, and contraction of concrete and elastic deformations of the foundations and abutments in a dam, are analyzed.

10.—A partly analytical and partly graphical design method is developed for the design of a compensated arch dam, which takes into full consideration the shape of the gorge at the dam site and in which the construction conditions are considered by insuring that the vertical joints of the loaded dam are under uniform compression.

III.—General Equations of the Horizontal Arch Section Under Constant Temperature, Based on Static Conditions

The radial loads applied along a horizontal arch are not constant, but vary, depending on the topography of the dam site and the shapes of the vertical and horizontal elements of the dam. No general rule covering the manner in which the loads will vary, can be made; it can only be stated that the arch loads will be symmetrical for symmetrical canyons and unsymmetrical for unsymmetrical canyons and foundation conditions. The problem of finding the proper shape of the arch resolves itself into finding the equation of the center-line curve of the arch for which the changing radial loads applied at the up-stream face will produce zero bending moments at every point of the center-line curve. Such an arch is herein termed a "compensated arch."

Referring to Fig. 3, let a horizontal element of the dam (height $= 1$) be fixed at the left abutment, and let the abutment at the right be replaced by the reaction, T_a, in which the subscript, a, indicates the values of the different functions at the abutments. Since no moment is to appear at the center line of the arch, this reaction must be applied tangentially at the center line and is equal and opposite to the arch thrust.

If the elastic center method is applied, the right end of the arch (Fig. 3) is connected by a rigid bracket to the elastic center, and the reaction, T_a, is replaced by forces and moments applied at that center.

Let the forces and moments that are applied at the elastic center be divided into forces and moments due to the external loads and internal

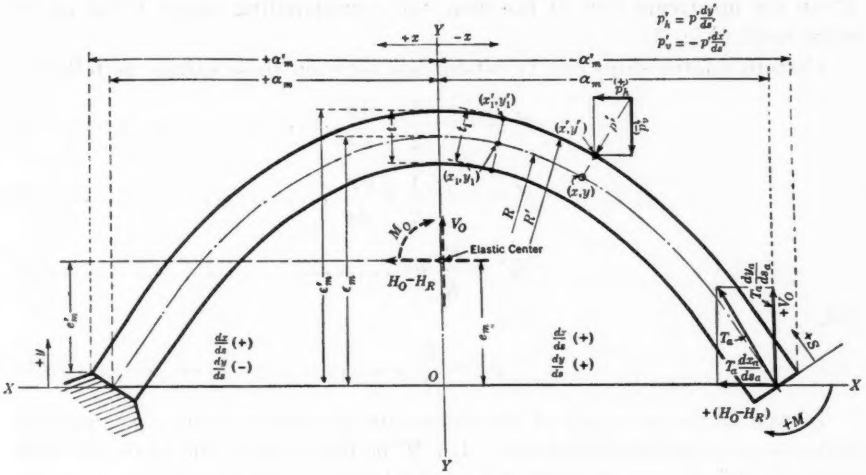


FIG. 3

stresses. Assuming for the present a symmetrical arch and resolving these forces in their horizontal and vertical components, let,

$H_0 =$ the horizontal component due to external loads;
$H_R =$ the horizontal component due to internal stresses;
$V_0 =$ the vertical component due to external loads;
$V_R =$ the vertical component due to internal stresses which is equal to zero for a symmetrical arch;
$M_0 =$ the moment due to external loads; and,
$M_R =$ the moment due to internal stresses which is equal to zero for a symmetrical arch.

Then, $(H_0—H_R)$ and V_0 and M_0 are the total horizontal and vertical components and total moment, respectively, applied at the elastic center of a symmetrical arch due to all external loads and internal stresses. Let the Y-axis run through the elastic center and let,

x and $y =$ the ordinates of the center line of the arch section;
$t =$ the radial thickness of the arch section;
$e_m =$ the distance of the elastic center from the X-axis;
$\pm\alpha_m =$ the limiting values of x;

s_m = the limiting values of y;
T = the total thrust at the center line of the arch section;
s = the length of the center line of the arch section. The origin of s is assumed at the right end of the arch, and the direction of s is assumed positive from right to left; and,
R = the radius of curvature of the center line curve of the arch section.

Since, in general, the thickness of the section, t, is not constant, the up-stream face of the dam upon which the loads, p', are applied, must be treated as an independent variable with respect to the center line of the section. In order to conform to this fact, the values of x, y, s, α_m, e_m, s_m, p, and R of the center line have corresponding values of x', y', s', α'_m, e'_m, s'_m, p', and R' on the up-stream face of the dam, the corresponding points being on the same radii (Fig. 3).

Certain relationships can be established between these values, as follows:

$$x' = x - \frac{1}{2}\, t\, \frac{dy}{ds} \qquad (1)$$

$$y' = y + \frac{1}{2}\, t\, \frac{dx}{ds} \qquad (2)$$

$$ds' = \frac{R'}{R}\, ds \qquad (3)$$

and,

$$p' = \frac{R}{R'}\, p \qquad (4)$$

In taking the moments of the forces about a point, as (x_1, y_1), positive moments are considered clockwise. Let M_1 be the moment due to the external loads, p', and M_2, the moment due to the forces at the elastic center of the arch.

From the definition of a compensated arch:

$$M_1 + M_2 = 0 \qquad (5)$$

Keeping in mind the positive and negative values of x', y', s', and p', indicated in Fig. 3, the loads, p', may be resolved into their horizontal and vertical components:

$$p'_h = +\, p'\, \frac{dy'}{ds'} \qquad (6)$$

and,

$$p'_v = -\, p'\, \frac{dx'}{ds'} \qquad (7)$$

Furthermore, the reaction, T_a, may be resolved into its vertical and horizontal components. It is to be noticed that these components are, respectively: $T_a \dfrac{dy_a}{ds_a} = V_0$, and $T_a \dfrac{dx_a}{ds_a} = (H_0 - H_R)$.

The moment, m_1, due to the single force, p' applied at the point, $(x'; y')$, with reference to the point, $(x_1; y_1)$, is:

$$m_1 = -p'_v\ (x - x') + p'_h\ (y - y')$$

and M_1, being the total moment at the point, $(x_1; y_1)$, of all the external forces, p', from the right abutment and up to the point, $(x_1; y_1)$, is expressed by $M_1 = \int_0^{s'_1} m_1\, ds'$.

Substituting Equation (6) and Equation (7):

$$M_1 = \int_{-a'm}^{x'_1} p'\ (x_1 - x')\ dx' + \int_{o'}^{y'_1} p'\ (y_1 - y')\ dy' \ldots\ldots\ldots (8)$$

in which, o′ is the value of y' at the right abutment.

Taking the moment due to the forces at the elastic center:

$$M_2 = M_O + (H_O - H_R)\ (y_1 - e_m) - V_O\, x_1 \ldots\ldots\ldots (9)$$

and, when $y_1 = 0, x_1 = -\alpha_m; M_2 = 0; M_O + (H_O - H_R)\ (-e_m) + V_O\, \alpha_m = 0$; or,

$$M_O = (H_O - H_R)\ e_m - V_O\, \alpha_m \ldots\ldots\ldots (10)$$

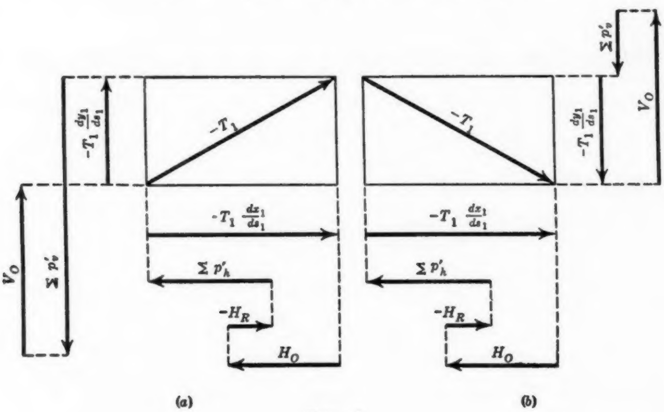


FIG. 4

Substituting Equation (10) in Equation (9), and collecting terms:

$$M_2 = (H_O - H_R)\ y_1 - V_O\ (\alpha_m + x_1) \ldots\ldots\ldots (11)$$

From Fig. 4,

$$(H_O - H_R) + \Sigma\, p'_h - T_1 \frac{dx_1}{ds_1} = 0 \ldots\ldots\ldots (12)$$

and,

$$V_O + \Sigma\ p'_v - T_1 \frac{dy_1}{ds_1} = 0 \ldots\ldots\ldots (13)$$

Since $p'_h = p' \dfrac{dy'}{ds'}$, and $\Sigma\, p'_h = \int_{o'}^{s'_1} p'_h\, ds'$, therefore, $\Sigma\, p'_h = + \int_{o'}^{y'_1} p'\, dy'$.

Likewise, $\Sigma\, p'_v = - \int_{-a'm}^{x'_1} p'\, dx'$.

Substituting these values in Equations (12) and (13), and re-arranging:

$$\int_{o'}^{y'_1} p'\, dy' = T_1 \frac{dx_1}{ds_1} - (H_0 - H_R) \text{ ....(14)}$$

and,

$$\int_{-a'm}^{x'_1} p'\, dx' = -T_1 \frac{dy_1}{ds_1} + V_0 \text{ ....(15)}$$

Differentiating Equation (5) with respect to s_1,

$$\frac{dM_1}{ds_1} + \frac{dM_2}{ds_1} = 0 \text{ ....(16)}$$

Differentiating Equation (8) with respect to s_1, substituting Equations (14) and (15), and re-arranging:

$$\frac{dM_1}{ds_1} = V_0 \frac{dx_1}{ds_1} - (H_0 - H_R) \frac{dy_1}{ds_1} + p'_1 (x_1 - x'_1) \frac{dx'_1}{ds_1} + p'_1 (y_1 - y'_1) \frac{dy'_1}{ds_1} \text{ ..(17)}$$

Differentiating Equation (11) with respect to s_1,

$$\frac{dM_2}{ds_1} = (H_0 - H_R) \frac{dy_1}{ds_1} - V_0 \frac{dx_1}{ds_1} \text{ ....(18)}$$

Substituting Equations (17) and (18) in Equation (16) and cancelling terms:

$$(x_1 - x'_1) \frac{dx'_1}{ds_1} + (y_1 - y'_1) \frac{dy'_1}{ds_1} = 0 \text{ ....(19)}$$

From Equations (1) and (2):

$$x_1 - x'_1 = + \frac{1}{2} t_1 \frac{dy_1}{ds_1} \text{ ....(20)}$$

and,

$$y_1 - y'_1 = - \frac{1}{2} t_1 \frac{dx_1}{ds_1} \text{ ....(21)}$$

Differentiating Equations (20) and (21) with respect to s_1 and re-arranging:

$$\frac{dx'_1}{ds_1} = \frac{dx_1}{ds_1} - \frac{1}{2} \frac{dt_1}{ds_1} \frac{dy_1}{ds_1} - \frac{1}{2} t_1 \frac{d^2y_1}{ds^2_1} \text{ ....(22)}$$

and,

$$\frac{dy'_1}{ds_1} = \frac{dy_1}{ds_1} + \frac{1}{2} \frac{dt_1}{ds_1} \frac{dx_1}{ds_1} + \frac{1}{2} t_1 \frac{d^2x_1}{ds^2_1} \text{ ....(23)}$$

From the calculus:

$$\frac{d^2y_1}{ds^2_1} = - \frac{1}{R_1} \frac{dx_1}{ds_1} \text{ ....(24)}$$

and,

$$\frac{d^2x_1}{ds^2_1} = + \frac{1}{R_1} \frac{dy_1}{ds_1} \text{ ....(25)}$$

Substituting Equation (24) in Equation (22) and re-arranging:

$$\frac{dx'_1}{ds_1} = \frac{dx_1}{ds_1}\left(1 + \frac{1}{2}\,\frac{t_1}{R_1}\right) - \frac{1}{2}\,\frac{dt_1}{ds_1}\,\frac{dy_1}{ds_1} \;\ldots\ldots\ldots\ldots(26)$$

Likewise, from Equation (23),

$$\frac{dy'_1}{ds_1} = \frac{dy_1}{ds_1}\left(1 + \frac{1}{2}\,\frac{t_1}{R_1}\right) + \frac{1}{2}\,\frac{dt_1}{ds_1}\,\frac{dx_1}{ds_1} \ldots\ldots\ldots\ldots(27)$$

Substituting Equations (20), (21), (26), and (27) in Equation (19), and simplifying,

$$\frac{dt_1}{ds_1} = 0 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(28)$$

which shows that the thickness of the arch section at a given elevation is constant.

Differentiating Equations (14) and (15) with respect to s_1:

$$p'_1\,\frac{dy'_1}{ds_1} = T_1\,\frac{d^2x_1}{ds^2_1} + \frac{d\,T_1}{ds_1}\,\frac{dx_1}{ds_1} \ldots\ldots\ldots\ldots\ldots\ldots(29)$$

and,

$$p'_1\,\frac{dx'_1}{ds_1} = -\;T_1\,\frac{d^2y_1}{d\,s^2_1} - \frac{d\,T_1}{ds_1}\,\frac{dy_1}{d\,s_1} \;\ldots\ldots\ldots\ldots\ldots\ldots(30)$$

Substituting Equation (22) in Equation (30), remembering Equation (28),

$$p'_1\left(\frac{dx_1}{d\,s_1} - \frac{1}{2}\,t\,\frac{d^2y_1}{ds^2_1}\right) = -\;T_1\,\frac{d^2y_1}{ds^2_1} - \frac{d\,T_1}{d\,s_1}\,\frac{dy_1}{d\,s_1} \;\ldots\ldots(31)$$

Substituting Equation (24) in Equation (31) and re-arranging:

$$p'_1 = \frac{\dfrac{T_1}{R_1}\,\dfrac{dx_1}{ds_1} - \dfrac{d\,T_1}{ds_1}\,\dfrac{dy_1}{ds_1}}{\dfrac{dx_1}{ds_1}\left(1 + \dfrac{1}{2}\,\dfrac{t}{R_1}\right)} \;\ldots\ldots\ldots\ldots\ldots\ldots(32)$$

Likewise, from Equation (29):

$$p'_1 = \frac{\dfrac{T_1}{R_1}\,\dfrac{dy_1}{d\,s_1} + \dfrac{d\,T_1}{ds_1}\,\dfrac{dx_1}{ds_1}}{\dfrac{dy_1}{d\,s_1}\left(1 + \dfrac{1}{2}\,\dfrac{t}{R_1}\right)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(33)$$

Equating Equations (32) and (33) and simplifying:

$$\frac{d\,T_1}{d\,s_1} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(34)$$

which further shows that the thrust at a given elevation is constant.

Substituting Equation (34) in Equation (33) and re-arranging: $T = p'_1\,(R_1 + \frac{1}{2}\,t)$. Since t is constant, $R'_1 = R_1 + \frac{1}{2}\,t$ (see Fig. 3), and, therefore,

$$T = p'_1\,R'_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(35)$$

or, substituting Equation (4),

$$T = p_1\,R_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(36)$$

Summarizing, an independent, symmetrical, compensated, horizontal arch has the following properties:

(*a*) The thickness, t is constant;
(*b*) The product of the up-stream radius, R', and the corresponding radial load, p', is equal to the product of the center-line radius, R, and the load on the center line, p, and is constant at any point of the arch;
(*c*) The thrust applied at the center line, T, is constant and equal to the product, $p\,R$ or $p'\,R'$; and,
(*d*) Since the thickness, t, is constant the elastic center is the center of gravity of the center line of the arch.

The same conclusions could be derived in a simpler way by making the assumption that the thickness, t, of the arch ring is constant. However, by making such an assumption no positive proof can be obtained that the arch ring with constant thickness is the only one that is relieved from bending moments, since the possibility would be open that an arch ring with varying thicknesses would be satisfactory.

By attacking the problem in this manner, the analysis proves positively that arch rings meeting the foregoing specifications are the only ones in which there are no bending moments.

IV.—Theory of Elasticity as Applied to an Independent Horizontal Compensated Arch at Constant and Varying Temperatures

The conditions developed in Section III must be met by every compensated arch, so that the static requirements are satisfied. In addition, the compensated arch must meet certain conditions to satisfy the requirements of the theory of elasticity. The only variable that has not been determined for a symmetrical arch is H_R, the horizontal component of the unbalanced part of the force applied at the elastic center. This variable does not depend on static considerations, but on the elastic properties of the material of the arch, on the coefficient of thermal expansion of this material, on the way in which the abutments are displaced under load, and on the difference in temperature of the arch material at the time when the arch is built and at any subsequent time under consideration.

Assume an arch under constant temperature conditions. Let the width equal unity and the cross-sectional area, t; then the relative deflection of two points in any desired direction is:

$$\Delta = \int_{s_1}^{s_2} \frac{1}{E}\,\frac{T}{t}\,n\,ds + \int_{s_1}^{s_2} \frac{M}{I}\,\frac{1}{E}\,m\,ds + \int_{s_1}^{s_2} \frac{V}{t}\,\frac{v}{E_s}\,C_s\,ds \dots\dots (37)$$

in which, T, M, and V are the total thrust, moment, and shear, respectively, at any point on the center line of the arch; n, m, and v are the thrust, moment, and shear, respectively, at any point on the center line of the arch, due to a unit load applied at the point, s_1, on the center line in the direction of the desired deflection; t is the thickness of the arch; E, the modulus of elasticity of the material in flexure; E_s, the modulus of elasticity of the material in shear; I, the moment of inertia of the cross-section about the center line; C_s, a constant depending upon the shape of the cross-section

(1.2 for rectangular sections); and s_1 and s_2 are the distances of the two points under consideration from a third point, as measured along the center line. The third point is assumed at the intersection of the center line of the arch and the right abutment (Fig. 5).

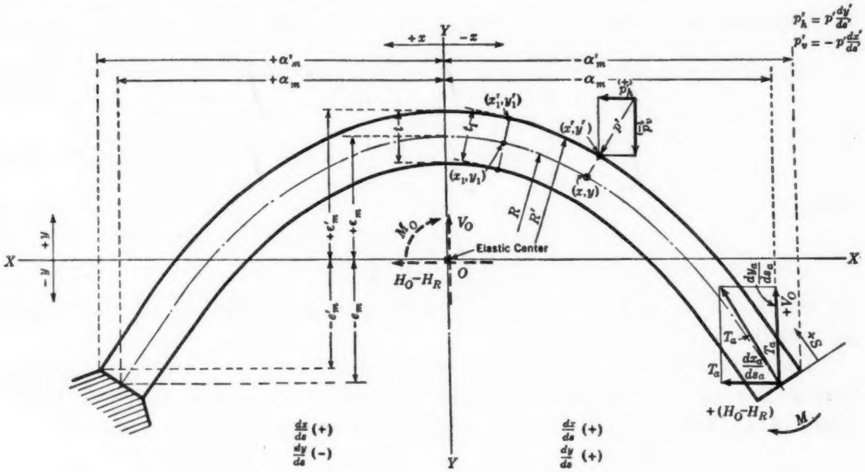


FIG. 5.

Making the elastic center and the elastic axes, the origin and axes of a co-ordinate system, the displacements of the point, s_1, are:

$$\Delta_x = \int_{s_1}^{S} \frac{T}{t}\,\frac{n_x}{E}\,ds + \int_{s_1}^{S} \frac{M}{I}\,\frac{m_x}{E}\,ds + \int_{s_1}^{S} \frac{V}{t}\,\frac{v_x}{E_s}\,C_s\,ds \;\ldots\ldots (38)$$

$$\Delta_y = \int_{s_1}^{S} \frac{T}{t}\,\frac{n_y}{E}\,ds + \int_{s_1}^{S} \frac{M}{I}\,\frac{m_y}{E}\,ds + \int_{s_1}^{S} \frac{V}{t}\,\frac{v_y}{E_s}\,C_s\,ds \;\ldots\ldots (39)$$

and,

$$\Delta_z = \int_{s_1}^{S} \frac{T}{t}\,\frac{n_z}{E}\,ds + \int_{s_1}^{S} \frac{M}{I}\,\frac{m_z}{E}\,ds + \int_{s_1}^{S} \frac{V}{t}\,\frac{v_z}{E_s}\,C_s\,ds \;\ldots\ldots (40)$$

in which,

S = total length of the center line of the arch;
Δ_x = linear displacement of the point, s_1, in the direction of the X-axis;
Δ_y = linear displacement of the point, s_1, in the direction of the Y-axis;
Δ_z = angular displacement; and,
$\left.\begin{matrix} n_x, m_x, v_x \\ n_y, m_y, v_y \\ n_z, m_z, v_z \end{matrix}\right\}$ = the corresponding thrust, moment, and shear due to the unit load applied at the elastic center.

Assuming a symmetrical arch as shown in Fig. 5,

$$n_x = -\frac{dx}{ds}\,;\;\; m_x = y\,;\;\; v_x = +\frac{dy}{ds}$$

$$n_y = -\frac{dy}{ds}\ ;\ m_y = -x\ ;\ v_y = +\frac{dx}{ds}$$

$$n_z = 0\ ;\ m_z = 1\ ;\ v_z = 0$$

Substituting in Equations (38), (39), and (40),

$$\Delta_x = -\int_{x_1}^{x_2} \frac{T}{t}\,\frac{1}{E}\,dx + \int_{s_1}^{S} \frac{M}{I}\,\frac{1}{E}\,y\,ds + \int_{y_1}^{y_2} \frac{V}{t}\,\frac{C_s}{E_s}\,dy \quad \ldots\ldots(41)$$

$$\Delta_y = -\int_{y_1}^{y_2} \frac{T}{t}\,\frac{1}{E}\,dy - \int_{s_1}^{S} \frac{M}{I}\,\frac{1}{E}\,x\,ds + \int_{x_1}^{x_2} \frac{V}{t}\,\frac{C_s}{E_s}\,dx \quad \ldots\ldots(42)$$

and,

$$\Delta_z = \int_{s_1}^{S} \frac{M}{I}\,\frac{1}{E}\,ds \quad \ldots\ldots\ldots\ldots\ldots\ldots(43)$$

in which, x_1, y_1, and x_2, y_2, are the corresponding ordinates of the points, s_1, and the left abutment.

The values of t, T, and I are constant and since for every point on the center line of the compensated arch M and V are equal to zero, Equations (41), (42), and (43) can be written (assuming that E is constant):

$$\Delta_x = -\frac{1}{E}\,\frac{T}{t}\,(x_2 - x_1)\,;\ \Delta_y = -\frac{1}{E}\,\frac{T}{t}\,(y_2 - y_1)\,;\ \text{and, } \Delta_z = 0$$

Following the previous assumption that the left abutment is fixed, the displacements, Δ_{xaR}, Δ_{yaR}, and Δ_{zaR}, of the right abutment can be obtained by substituting the lower and upper limits for the values, of x, y, and s, corresponding to the left and right abutments, respectively.

For the symmetrical arch (Fig. 5):

$$x_1 = +\,a_m\,;\ x_2 = -\,a_m$$

$$y_1 = -\,e_m\,;\ y_2 = -\,e_m$$

Therefore,

$$\Delta_{xaR} = 2\,a_m\,\frac{1}{E}\,\frac{T}{t} \quad \ldots\ldots\ldots\ldots\ldots\ldots(44)$$

$$\Delta_{yaR} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots(45)$$

and,

$$\Delta_{zaR} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots(46)$$

In order to "compensate" the arch Equation (44) requires that the right abutment must be moved horizontally in the positive direction or toward the left abutment for the distance, $2\,a_m\,\frac{1}{E}\,\frac{T}{t}$, which would be opposite to the horizontal movement that would take place as a result of the displacement of the abutment due to the arch thrust.

Temperature Effects.—Considering the effect of the change of temperature, a drop in temperature is analogous to the effect of rib-shortening and the term, $\frac{1}{E}\,\frac{T}{t}$, in Equation (44) can be substituted by $c_T\,T_T$, in which c_T is the

coefficient of linear expansion of the material, and T_T is the temperature change, in degrees, being positive for a drop in temperature, and negative for a rise in temperature.

Making this substitution in Equation (44), $\Delta'_{xaR} = 2\, \alpha_m\, c_T\, T_T$, which indicates that, in order to compensate the arch for temperature changes, the right abutment (Fig. 5) is moved horizontally in the positive direction for the distance, $2\, \alpha_m\, c_T\, T_T$. The total horizonal movement that would be required in the positive direction, considering rib-shortening and temperature change, is:

$$\Delta_{xaR} = 2\, \alpha_m \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right) \qquad \text{(47)}$$

If the abutments are considered as fixed, then $\Delta_{xaR} = 0$, and Equation (44) cannot be satisfied.

Consequently, a moment will appear in the arch; applying the theory of elasticity it can be shown that it is equal to a moment that is due to the force, H_R, applied at the elastic center. This moment will be $M_R = -H_R\, y$.

Neglecting V and substituting M_R for M in Equation (41), remembering Equation (47):

$$\Delta_{xaR} = 2\, \alpha_m \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right) - \frac{H_R}{I}\,\frac{1}{E}\int_0^S y^2\, d\, s = 0 \qquad \text{(48)}$$

and,

$$H_R = 2\, \alpha_m \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right) \frac{I\,E}{\int_0^S y^2\, d\, s} \qquad \text{(49)}$$

If the abutments are moved the value of H_R will be increased or decreased and, for the movement of the abutments equal to $2\, \alpha_m \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right)$, the force, H_R, will be equal to zero and the arch will be compensated.

If the displacements of the abutments under loading are considered and if the horizontal components of the displacements of the right and left abutments are D_{haR} and D_{haL}, respectively, then for the purpose of this analysis it can be assumed that the left abutment remains fixed, and the right abutment is displaced in the horizontal direction for the distance,

$$\Delta_{xaR} = -\,(D_{haL} - D_{haR})$$

Substituting this value in Equation (48),

$$2\, \alpha_m \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right) - \frac{H_R}{I}\,\frac{1}{E}\int_0^s y^2\, d\, s = -\,(D_{haL} - D_{haR})$$

Whence,

$$H_R = \frac{I\,E}{\int_0^s y^2\, d\, s}\left[2\, \alpha_m \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right) + (D_{haL} - D_{haR})\right]$$

The force, H_R, can be divided into three parts, $H_{RS} + H_{RT} + H_{RA}$, in which,

$$H_{RS} = 2\,\alpha_m \frac{T}{t} \frac{I}{\int_0^S y^2\,d\,s} \qquad (50)$$

$$H_{RT} = 2\alpha_m\, c_T\, T_T \frac{I\,E}{\int_0^S y^2\,d\,s} \qquad (51)$$

and,

$$H_{RA} = (D_{haL} - D_{haR}) \frac{I\,E}{\int_0^S y^2\,d\,s} \qquad (52)$$

Equation (50) is the horizontal component of the force applied at the elastic center due to the elastic deformations of the material of the dam; Equation (51) is a similar force due to the changes of temperature of the material of the dam; and Equation (52) is a similar force due to the displacements of the abutments. The vertical components of all these forces are equal to zero for a symmetrical arch.

In order to compensate the arch, the right abutment is moved in the positive direction for the distance, $\Delta_{xaR} = 2\,\alpha_m \left(\frac{1}{E}\frac{T}{t} + c_T\,T_T\right) + (D_{haL} - D_{haR})$, which will make H_R, and, consequently, M_R, equal to zero.

Some other means can be provided that will make H_R equal to zero and, as far as the arch is concerned, will give the same result as a movement of the abutment. This cannot be done by simple changes in the shape of the arch, since the shape is determined from the static requirements.

The important conclusion to be derived from the elastic equations is that the only way to relieve the arch from bending moments is to shape it so as to conform with the conditions derived from statical considerations and to create conditions under which the force, H_R, due to rib-shortening, abutment deflections, and difference in temperature, is made equal to zero.

V.—Compensation of the Independent Horizontal Arch

One way to compensate the properly shaped arch for the moments, M_R, is to introduce stresses that are equal and opposite to those due to the moment, M_R. Mathematically, this is expressed by a force applied at the elastic center of the arch equal and opposite to H_R, neutralizing the action of H_R and compensating the arch in the same way as in the case of a movement of the abutment. It has been proposed to introduce such stresses by proper pressure grouting of the vertical construction joints, but this is open to certain objections.

A positive method of compensating the arch is to provide notches as illustrated in Fig. 6(a). The widths of the notches are extremely exaggerated

to make the diagram clear. These notches can be designed so as to compensate the arch for the effect of rib-shortening, change in temperature from the time of closing the dam to the average temperature which the dam will attain in service, and for displacement of the abutments.

In an arch compensated in this manner, the deflections will differ from those in an uncompensated arch, and are due to two factors: First, the closing of the notches; and, second, the deformation due to axial compression.

Fig. 6(*b*) illustrates a section of the compensated arch adjacent to the right abutment. The notches in the down-stream face, as shown, will be

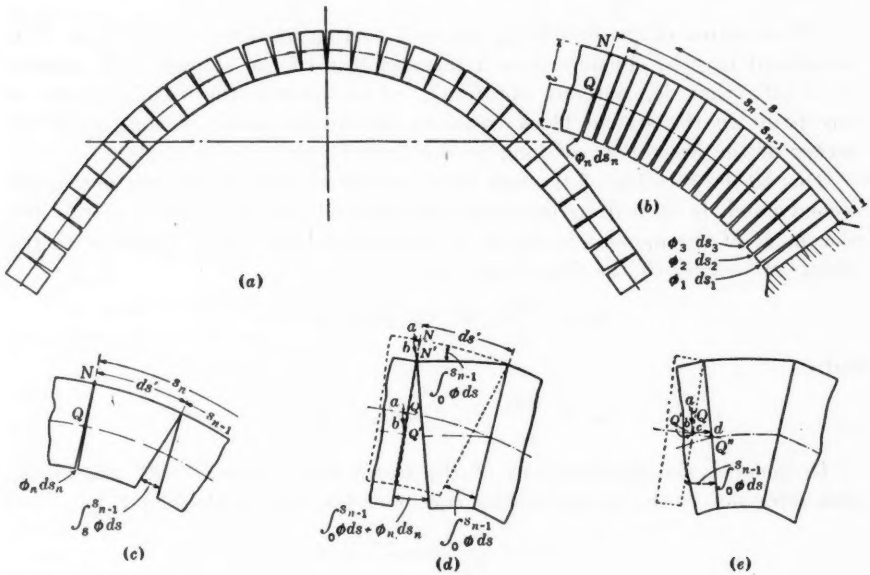


FIG. 6.

assumed positive, those in the up-stream face, near the crown, being assumed negative. The directions of the horizontal and vertical deflections are assumed positive in the positive directions of x and y, respectively.

Let ϕ be the angular measure of a notch per unit of length in the center of the arch. Then the angle of each notch will be $\phi\ ds$. The width of any notch at the face of the arch will be,

$$\Delta_b = t\ \phi\ ds \qquad (53)$$

Starting at the right abutment, let the angle of the first infinitely small notch be $\phi_1\ ds_1$ and that of the second, $\phi_2\ ds_2$ (Fig. 6(*b*)). As the first notch closes, the second one will open by the same angular amount, and, as long as the angles are small, the result will be $\phi_1\ ds_1 + \phi_2\ ds_2$. In turn, when the second notch is also closed the third notch, with a normal opening of $\phi_3\ ds_3$, will open by the amount the second one closed, or the resulting angle will be $\phi_1\ ds_1 + \phi_2\ ds_2 + \phi_3\ ds_3$.

Reducing the widths of the blocks until they are infinitely small, the resulting angle at s_{n-1}, for instance, after all the notches to the right of it have closed, will be $\int_0^{s_{n-1}} \phi\, ds$. When the first notch, with an opening, $\phi_1\, ds_1$, closes, the block between it and the next notch turns at the same angle, rotating about the upper right corner. The horizontal and vertical components of the displacement of the upper left corner, due to this rotation, are:

$$\delta_{h1} = \frac{dy_1}{ds_1}\, ds'_1\, (\phi_1\, ds_1); \text{ and } \delta_{v1} = \frac{dx_1}{ds_1}\, ds'_1\, (\phi_1\, ds_1)$$

The rotation of the first block causes a movement of the second block. This movement may be considered as a transposition of the second block parallel to itself (that is, without rotation), which makes the displacement of any point in the second block equal to the displacement of the upper left corner of the first block, as long as the second notch does not close.

Due to the rotation of the first block, or the closing of the first notch, the second notch is opened, as previously stated, and the components of the displacement of the upper left corner of the second block, with reference to the upper left corner of the first block, are:

$$\delta_{h2} = \frac{dy_2}{ds_2}\, ds'_2\, (\phi_1\, ds_1 + \phi_2\, ds_2)$$

and,

$$\delta_{v2} = \frac{dx_2}{ds_2}\, ds'_2\, (\phi_1\, ds_1 + \phi_2\, ds_2)$$

In general, the displacement of the upper left corner of the nth block, with reference to the corresponding corner of the nth—1 block, will be:

$$\delta_{h2} = \frac{dy_n}{ds_n}\, ds'_n\, \Sigma^n{}_1\, \phi\, ds$$

and,

$$\delta_{vn} = \frac{dx_n}{ds_n}\, ds'_n\, \Sigma^n{}_1\, \phi\, ds$$

The total horizontal and vertical components of the displacement of the upper left corner of the nth block (designated as N in Fig. 6(b)), can be expressed as the sum of the horizontal and vertical displacements of the upper left corners of all the preceding blocks with reference to the corresponding corner of each preceding block, thus: $a' = \Sigma^n{}_1\, \delta_n$.

By decreasing the widths of the blocks until they are infinitely small, the following integral expressions for the displacement of the point from N to N' (Fig. 6(d)) may be written:

$$a = \int_0^{s_{n-1}} \frac{dy}{ds}\, ds' \int_0^{s} \phi\, ds; \text{ and, } b = \int_0^{s_{n-1}} \frac{dx}{ds}\, ds' \int_0^{s} \phi\, ds$$

which are also the displacements of the point from Q to Q' as indicated in Fig. 6(d).

After the notch between the nth $-$ 1 and nth blocks has closed, the resulting notch between the nth and nth + 1 blocks will be: $\int_0^{s_n} \phi\, ds = \int_0^{s_{n-1}} \phi\, ds + \phi_n\, ds_n$, which may be considered equal to $\int_0^{s_{n-1}} \phi\, ds$ since both ϕ_n and ds_n are infinitely small. In order to close the arch, this notch must close independently of the preceding notches, and its closing displaces the point on the center line of the nth + 1 block from Q' to Q'', as shown in Fig. 6(e). The horizontal and vertical components of this displacement are:

$$c = \frac{1}{2}\, t\, \frac{dx}{ds} \int_0^{s_{n-1}} \phi\, ds; \text{ and, } d = \frac{1}{2}\, t\, \frac{dy}{ds} \int_0^{s_{n-1}} \phi\, ds$$

After all the notches are closed, the center line of the arch is shortened by the axial compression and shortened or lengthened by the decrease or increase in temperature. The horizontal and vertical components of the shortening of the center line, neglecting the widths of the notches, are:

$$\delta_h = \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right) \int_0^{s_{n-1}} \frac{dx}{ds}\, ds; \text{ and, } \delta_v = \left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right) \int_0^{s_{n-1}} \frac{dy}{ds}\, ds$$

The total horizontal and vertical components of the displacement of the point from Q to Q'' are:

$$D_h = +\,a - c - \delta_h \qquad (54)$$

and,

$$D_v = -\,b - d - \delta_v \qquad (55)$$

Due to the difference in the lengths of the down-stream face, in which the total width of the notch occurs, and the center line, in which only one-half the width of the notch appears, the uniform thrust over the section will cause a differential shortening between the down-stream face and the center line. This differential shortening may be conceived as a compressing together of the faces of the notch before it is closed, due to the thrust on the section, after which the notch is closed. This action of the notches under stress is a reversal of the sequence as compared with actual conditions in the arch.

Fig. 7 illustrates one-half a segment of the arch, of a length, ds, on the center line, with the open notch, $\phi\, ds$, shown on the right. The face of the notch, before and after the thrust is applied, is AB and $A'B'$, respectively. The angle between the radius, AO, and the face of the notch is $\frac{1}{2}\,\phi\, ds$.

Let the angle between the radii, AO and CO, be $\frac{1}{2}\, d\alpha$; the radius to the center line, R; and the radius to the down-stream face, R''. After the thrust is applied, AO will be moved to $A'O$ and the position of AB will be changed to $A'B'$. The angle between $A'O$ and $A'B'$ is designated as $\frac{1}{2}\,\beta\, ds$, which is the value of $\frac{1}{2}\,\phi\, ds$ after the faces of the notch are compressed together due to the action of the thrust, and which is to be substituted for $\frac{1}{2}\,\phi\, ds$ in the equations for the deflections of the arch. The angle between $A'O$ and AO will be $\left(\frac{1}{E}\,\frac{T}{t} + c_T\, T_T\right)\frac{1}{2}\, d\alpha$, considering change in length due to changes in temperature also.

Referring to Fig. 7:

$$DB + BE = DB' + B'E' + E'E \quad \text{.................(56)}$$

in which,

$$DB = \frac{1}{2}\, R''\, d\alpha - \frac{1}{2}\, t\, \phi\, ds$$

$$BE = \frac{1}{2}\, t\, \phi\, ds$$

$$DB' = \left(1 - \frac{1}{E}\frac{T}{t} - c_T\, T_T\right) DB = \frac{1}{2}\left(1 - \frac{1}{E}\frac{T}{t} - c_T\, T_T\right)(R''\, d\alpha - t\, \phi\, d\, s)$$

$$B'E' = \frac{1}{2}\, t\, \beta\, ds$$

and,

$$E'E = \left(\frac{1}{E}\, \frac{T}{t} + c_T\, T_T\right) \frac{1}{2}\ \ R''\, d\, \alpha$$

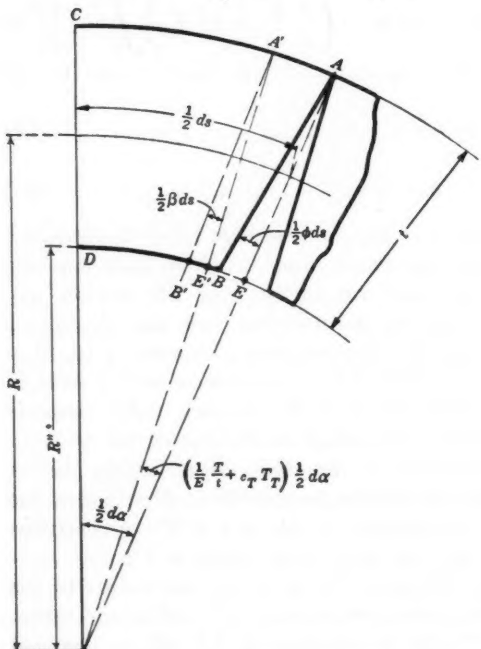


FIG. 7.

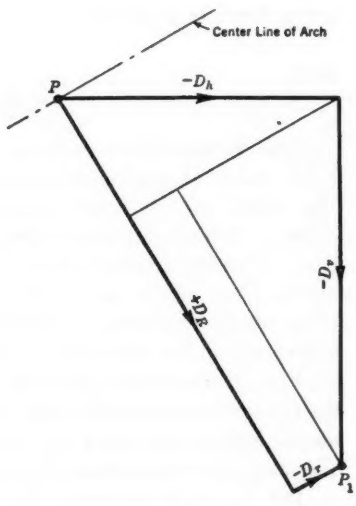


FIG. 8.

In other words, Equation (56) equals,

$$R''\, d\alpha - t\, \phi\, ds + t\, \phi\, ds = R''\, d\alpha - t\, \phi\, ds$$

$$-\left(\frac{1}{E}\, \frac{T}{t} + c_T\, T_T\right)\left(R''\, d\alpha - t\, \phi\, ds\right) + t\, \beta\, ds + \left(\frac{1}{E}\, \frac{T}{t} + c_T\ T_T\right) R''\, d\alpha$$

Whence,

$$\beta = \left(1 - \frac{1}{E}\frac{T}{t} - c_T\, T_T\right)\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

An identical solution is arrived at by considering the notches in the upstream face.

If a displacement of the abutments occurs, an addition to the angle of the notch, $\phi\, ds$, must be made. The displacements of each abutment can be resolved into the vertical components, D_{vaL} and D_{vaR}, and the horizontal components, D_{haL} and D_{haR}, in which, the subscript, aL, refers to the left abutment, and aR to the right abutment.

Noting that $ds' = \dfrac{R'}{R} ds$ and adding the residual displacements of the abutment, the equations for the horizontal and vertical components of the deflections of the arch may be written from Equations (54) and (55), as follows:

$$D_h = \int_0^{y_{n-1}} \frac{R'}{R}\, dy \int_0^s \beta\, ds - \frac{1}{2}\, t\, \frac{dx}{ds} \int_0^s \beta\, ds$$
$$- \left(\frac{1}{E}\frac{T}{t} + c_T\, T_T\right) \int_{-a_m}^{x_{n-1}} dx + D_{haR} \quad \ldots\ldots\ \ldots\ldots\ldots (58)$$

and,

$$D_v = - \int_{-a_m}^{x_{n-1}} \frac{R'}{R}\, dx \int_0^s \beta\, ds - \frac{1}{2}\, t\, \frac{dy}{ds} \int_0^s \beta\, ds$$
$$- \left(\frac{1}{E}\frac{T}{t} + c_T\, T_T\right) \int_0^{y_{n-1}} dy + D_{vaR} \quad \ldots\ldots\ldots\ldots\ldots (59)$$

Differentiating Equation (58) with respect to s:

$$\frac{d\,D_h}{ds} = \frac{R'}{R}\frac{dy}{ds}\int_0^s \beta\, ds - \frac{1}{2}\, t\, \frac{d^2 x}{ds^2}\int_0^s \beta\, ds - \frac{1}{2}\, t\, \frac{dx}{ds}\,\beta - \left(\frac{1}{E}\frac{T}{t} + c_T\, T_T\right)\frac{dx}{ds}$$

Multiplying by ds, substituting Equation (25), and collecting terms:

$$dD_h = dy \int_0^s \beta\, ds - \left(\frac{1}{2} t\, \beta + \frac{1}{E}\frac{T}{t} + c_T\, T_T\right) dx \quad \ldots\ldots\ldots (60)$$

A similar solution of Equation (59) gives:

$$d\,D_v = -\, dx \int_0^s \beta\, ds - \left(\frac{1}{2} t\, \beta + \frac{1}{E}\frac{T}{t} + c_T\, T_T\right) dy \quad \ldots\ldots (61)$$

Integrating Equations (60) and (61):

$$D_h = \int dy \int_0^s \beta\, ds - \int \left(\frac{1}{2} t\, \beta + \frac{1}{E}\frac{T}{t} + c_T\, T_T\right) dx + c_h \quad \ldots (62)$$

and,

$$D_v = - \int dx \int \beta\, ds - \int \left(\frac{1}{2} t\, \beta + \frac{1}{E}\frac{T}{t} + c_T\, T_T\right) dy + c_v \ldots (63)$$

in which, c_h and c_v are integration constants.

When $s = 0$, the horizontal deflection is equal to the deflection of the right abutment. Referring to Equation (63):

$$D_{haR} - D_{haL} = \int_0^0 dy \int_0^0 \beta\, ds - \int_{-am}^{-am} \left(\frac{1}{2}\, t\, \beta + \frac{1}{E}\, \frac{T}{t} + c_T\, T_T\right) dx + c_h = c_h$$

Similarly, $D_{vaR} - D_{vaL} = c_v$. Substituting in Equations (62) and (63):

$$D_h = \int_0^y dy \int_0^s \beta\, ds - \int_{-am}^{x} \left(\frac{1}{2}\, t\, \beta + \frac{1}{E}\, \frac{T}{t} + c_T\, T_T\right) dx + D_{haR} - D_{haL}$$

and,

$$D_v = - \int_{-am}^{x} dx \int_0^s \beta\, ds - \int_0^y \left(\frac{1}{2}\, t\, \beta + \frac{1}{E}\, \frac{T}{t} + c_T\, T_T\right) dy + D_{vaR} - D_{vaL}$$

The radial deflection, D_R, and the tangential deflection, D_T, of a point on the center line of the arch may be found from the values of D_h and D_v.

Fig. 8 shows the deflection of a point, P, on the center line of the arch, to the position, P_1. Remembering that $\frac{dy}{ds}$ is negative and $\frac{dx}{ds}$ is positive on the left side, the following equations may be written:

$$D_R = - D_v \frac{dx}{ds} + D_h \frac{dy}{ds} \qquad \text{(64)}$$

and,

$$D_T = + D_h \frac{dx}{ds} + D_v \frac{dy}{ds} \qquad \text{(65)}$$

in which, D_R is positive in the down-stream direction, and D_T is positive in the positive direction of s.

Differentiating Equation (64) with respect to s,

$$\frac{d D_R}{ds} = - D_v \frac{d^2 x}{d s^2} + D_h \frac{d^2 y}{d s^2} - \frac{d D_v}{ds} \frac{dx}{ds} + \frac{d D_h}{ds} \frac{dy}{ds} \qquad \text{(66)}$$

Substituting Equations (24) and (25) in Equation (66), and re-arranging:

$$\frac{d D_R}{ds} = - \frac{1}{R} \left(D_h \frac{dx}{ds} + D_v \frac{dy}{ds}\right) - \frac{d D_v}{ds} \frac{dx}{ds} + \frac{d D_h}{ds} \frac{dy}{ds}$$

Referring to Equation (65), substituting Equations (60) and (61), and simplifying:

$$\frac{d D_R}{ds} = - \frac{D_T}{R} + \int_0^s \beta\, ds \qquad \text{(67)}$$

A similar solution from Equation (65) gives,

$$\frac{d D_T}{ds} = \frac{D_R}{R} - \frac{1}{2}\, t\, \beta - \frac{1}{E}\, \frac{T}{t} - c_T\, T_T \qquad \text{(68)}$$

Substituting the limits of the right and left abutments in Equation (67),

$$\left(\frac{dD_R}{ds}\right)_{aR} = - \left(\frac{D_T}{R}\right)_{aR} + \int_0^0 \beta\, ds; \text{ and} \left(\frac{dD_R}{ds}\right)_{aL} = - \left(\frac{D_T}{R}\right)_{aL} + \int_0^S \beta\, ds$$

Since $\int_0^{\circ} \beta\, ds = 0$ at the right abutment, $\left(\frac{dD_R}{ds}\right)_{aR} + \left(\frac{D_T}{R}\right)_{aR} = 0$. Likewise, $\left(\frac{dD_R}{ds}\right)_{aL} + \left(\frac{D_T}{R}\right)_{aL} = 0$ for a symmetrical arch, and, therefore,

$$\int_0^S \beta\, ds = 0 \qquad (69)$$

Equation (68) can also be derived by equalizing the internal and external work. Neglecting the widths of the notches, the internal work done in the arch will be one-half the product of the thrust, T, times the deformation of the center line. The deformation of the center line is $\left(\frac{1}{E}\frac{T}{t} + c_T\, T_T\right)\int_0^S ds$.

The internal work done in the arch will be, $W_I = \frac{1}{2}\, T\left(\frac{1}{E}\frac{T}{t} + c_T\, T_T\right)\int_0^S ds$.

The external work done to the radial loads on the up-stream face is equal to one-half the sum of the products of the load at each point times the radial deflection at that point. From this must be deducted the external work done at the abutment. The external work due to the reaction, T, is one-half the product of the thrust times the tangential deflection at the abutment. This deflection is the tangential deflection on the center line of the arch plus the width of the notch on the center line at the abutment after all the notches toward the crown have been closed. This notch will be $\int_0^s \beta\, ds$, and the width on the center line will be $\frac{1}{2}\, t \int_0^s \beta\, ds$.

The equation for external work will be:

$$W_E = \frac{1}{2}\int_0^S p'\, ds'\, D_R - \frac{1}{2}\, T \int_0^S dD_T - \frac{1}{4}\, Tt \int_0^S \beta\, ds$$

Since $ds' = \frac{R'}{R}\, ds$ and $p'R' = T$:

$$W_E = \frac{1}{2}\, T \left[\int_0^S \frac{D_R}{R}\, ds - \int_0^S dD_T - \frac{1}{2}\, t \int_0^S \beta\, ds\right]$$

Equalizing the internal and external work:

$$\left(\frac{1}{E}\frac{T}{t} + c_T\, T_T\right)\int_0^S ds = \int_0^S \frac{D_R}{R}\, ds - \int_0^S dD_T - \frac{1}{2}\, t \int_0^S \beta\, ds$$

or,

$$\int_0^S dD_T = \int_0^S \frac{D_R}{R}\, ds - \frac{1}{2}\, t \int_0^S \beta\, ds - \left(\frac{1}{E}\frac{T}{t} + c_T\, T_T\right)\int_0^S ds \qquad (70)$$

Differentiating Equation (70) with respect to s, the result is a formula identical to Equation (68).

Dividing Equation (67) by Equation (68) and simplifying,

$$\frac{dD_R}{dD_T} = \frac{-D_T + R\int_0^s \beta\, ds}{D_R - R\left(\frac{1}{2}\,t\beta + \frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)}$$

Further simplification gives:

$$D_R\,dD_R + D_T\,dD_T = R\,dD_T\int_0^s \beta\,ds + R\,dD_R\,\left(\frac{1}{2}\,t\,\beta + \frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right).\;(71)$$

This differential equation gives the relationship between D_R and D_T.

A solution of Equation (71) is:

$$D_R = \left(\frac{1}{2}\,t\,\beta + \frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)R\;\ldots\ldots\ldots\ldots\ldots(72)$$

and,

$$D_T = R\int_0^s \beta\,ds\;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(73)$$

From Equation (72):

$$\beta = \frac{2}{t}\left[\frac{D_R}{R} - \left(\frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)\right]\ldots\ldots\ldots\ldots\ldots\ldots(74)$$

therefore,

$$\int \beta\,ds = \frac{2}{t}\int \frac{D_R}{R}\,ds - \frac{2}{t}\left(\frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)\int ds\;\ldots\ldots\ldots(75)$$

and,

$$\int_0^s \beta\,ds = \frac{2}{t}\int_0^s \frac{D_R}{R}\,ds - \frac{2}{t}\left(\frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)s\;\ldots\ldots\ldots(76)$$

From Equation (69):

$$\int_0^S \frac{D_R}{R}\,ds = \left(\frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)S\ldots\ldots\ldots\ldots\ldots\ldots(77)$$

Assuming a relationship between D_R and R, for instance,

$$D_R = \frac{\gamma}{R}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(78)$$

in which, γ is a constant and remembering that, $R = \frac{T}{p}$, Equations (74) and (76) can be written:

$$\beta = \frac{2}{t}\left[\frac{\gamma}{T^2}\,p^2 - \left(\frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)\right]\ldots\ldots\ldots\ldots\ldots\ldots(79)$$

$$\int_0^s \beta\,ds = \frac{2}{t}\,\frac{\gamma}{T^2}\int_0^s p^2\,ds - \frac{2}{t}\left(\frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)\ldots\ldots\ldots(80)$$

Remembering Equation (69):

$$\gamma = \left(\frac{1}{E}\,\frac{T}{t} + c_T\,T_T\right)\frac{T^2 S}{\int_0^S p^2\,ds}\;\ldots\ldots\ldots\ldots\ldots\ldots(81)$$

Introducing Equation (81) in Equation (79):

$$\beta = \frac{2}{t}\left(\frac{1}{E}\;\frac{T}{t} + c_T\,T_T\right)\left(\frac{p^2\,S}{\int_0^S p^2 ds} - 1\right) \dots\dots\dots\dots (82)$$

Finally, from Equations (72, (73), and (57):

$$D_R = \left(\frac{1}{E}\;\frac{T}{t} + c_T\;T_T\right)\;\frac{p\;T\;S}{\int_0^S p^2 ds}\;\dots\dots\dots\dots\dots (83)$$

$$D_T = \frac{2}{t}\left(\frac{1}{E}\;\frac{T}{t}\; + c_T\,T_T\right)\frac{T}{p}\left\{\frac{\int_0^s p^2 ds}{\int_0^S p^2 ds}\;S - s\right\}\;\dots\dots (84)$$

and,

$$\phi = \frac{2}{t}\;\frac{\frac{1}{E}\;\frac{T}{t} + c_T\,T_T}{1 - \left(\frac{1}{E}\;\frac{T}{t} + c_T\,T_T\right)}\left\{\frac{p^2\,S}{\int_0^S p^2\,ds} - 1\right\}\;\dots\dots\dots (85)$$

Equation (53) gives the value of the width of the notch at the face of the dam for an infinitely small notch. For all practical purposes, the notches may be constructed at commensurate intervals, and their widths can be computed from the value of β for the mid-point of the length of the center line of the arch between the notches, which may be called s_N. This may be done as long as the notches are small enough to allow the arch to deflect within the elastic limit of the material.

Substituting s_N for ds in Equation (53) and remembering Equation (57):

$$\Delta_b = 2\,s_N\;\frac{\frac{1}{E}\;\frac{T}{t} + c_T\,T_T}{1 - \left(\frac{1}{E}\;\frac{T}{t} + c_T\;T_T\right)}\left\{\frac{p^2\,S}{\int_0^S p^2\,ds} - 1\right\}$$

As far as the actual construction of the notches is concerned, one of the simplest methods is to place the concrete in alternate blocks and to provide the **V**-shaped joints, by means of elastic fillers placed between the adjoining blocks. During the time of loading the arch, the **V**-shaped joints will close and compress the fillers flat or squeeze them out. The difference between the present methods of design and the one proposed is that, at present, under zero load conditions the vertical joints have parallel faces and will be **V**-shaped after the load is applied. The proposed method will have **V**-shaped joints under zero load conditions and tightly compressed joints with parallel faces under full load.

The introduction of the **V**-shaped notches is a radical deviation from accepted design methods since the arbitrary assumption of a continuous horizontal arch is removed, and the arch is considered to be composed of a number

of arch blocks in conformity with actual construction practice. The sizes of the **V**-shaped joints that are computed in accordance with the foregoing formulas represent the minimum necessary to compensate the arch. If a larger deflection of the horizontal arches is desired to improve the stress conditions of the vertical elements, that can be accomplished by increasing the size of the **V**-shaped joints.

Another way to compensate the horizontal arches without actually building the **V**-shaped joints is to take advantage of the contraction of concrete after it is placed. If the blocks are properly shaped and alternately placed, the contraction of concrete will result either in the actual forming of **V**-shaped joints, or in the introduction in the concrete of internal tension stresses the action of which will be the same as if **V**-shaped joints are provided.

VI.—Vertical Elements

In most of the present arch dam designs, the shapes of the vertical elements are chosen arbitrarily and proper attention is seldom given to the influence which these shapes have on the general behavior of the dam, and the conditions at the horizontal joints and foundations before and after the water load is applied. The following investigation of the action of the vertical elements is made under the assumption of straight-line stress distribution at the horizontal joints and foundations.

Each horizontal joint, as well as the foundation, carries a vertical load that is closely equal to the sum of the dead weight of the part of the dam resting on the joint and the vertical component of the total water load. The arch action has only a negligible effect on this load. The load resting on the horizontal joints and on the foundation produces certain stresses, the distribution of which is different before and after the water load is applied, and both cases should be considered in the design.

During construction the vertical elements are built up as independent piers and the stresses at the horizontal joints and at the foundations, as well as the deflections of the vertical elements, depend on the dead weight of the concrete only as long as no water load is applied. If the vertical cross-sections were made symmetrical with reference to the vertical center line, then the center of gravity of each section would always fall at the middle of the base and under the condition of no water load, the distribution of stresses at each joint would be uniform. The deflection would appear in the vertical direction only, the horizontal component of the deflection being equal to zero. If the shape of the vertical section is not symmetrical, then the center of gravity will be shifted, the stress distribution at the joints will be non-uniform, and, in addition to the vertical deflection, a horizontal deflection will be introduced.

As long as the gravity resultant falls within the middle third of the base at every point of the vertical section, no tension will occur; if the resultant is outside the middle third, tension will appear, and if the resultant is outside the base, the section will be unstable and may overturn during construction unless some means of support are provided.

Considering each vertical element as an independent pier even after the dam is built and closed, and after the part of the water load that is taken by the vertical element is applied, the distribution of stresses in the horizontal joints and at the foundation will depend upon the relative location of the resultant of the dead weight and water load forces.

The distribution of stresses can be specified in any desirable way, the limit being that different distributions will occur for conditions of water load and no water load, which would preclude a specification requiring a uniform distribution of stresses under both conditions. Among the possible specifications for stress distribution at the horizontal joints and at the foundation, the following may be of interest:

1.—No tension stresses are permissible in the design, with reservoir empty, and for water load, the stress distribution must be uniform.

2.—No tension stresses for reservoir empty and, for water load, the compression at the up-stream face must be equal to, or greater than, the corresponding hydrostatic pressure in order to secure absolutely water-tight horizontal joints on the assumption that the horizontal construction joints would be water-tight under no-load conditions.

3.—No tension stresses are permissible under either water load conditions or reservoir empty.

4.—Reasonably low tension stresses are required under both conditions of loading.

The conventional designs of vertical dam sections cannot meet the first two specifications and generally the best that can be done with such designs is to have them meet the fourth specification. Specifications 1 and 2 require very special designs to make the sections safe under no-water load conditions. Specification 3 represents a reasonable compromise between the rigid requirements mentioned under Specifications 1 and 2 and the usual designs. In such sections a reversal of conditions under no-water load and under total water load must take place, the resultants in both cases intersecting the base within the middle third.

The study of the vertical element is most conveniently started at the crown of a symmetrical dam in which the combined gravity and arch action are most pronounced and the deflections are simplest. Considering such an element, it is necessary to realize the peculiarities of its behavior under water loading. At the lowest part of the dam, the total water pressure is resisted by the vertical elements with very small deflections. Disregarding the effect of Poisson's ratio and the deflections of the foundations, the arch element at the bottom of the dam has no load at all. At higher elevations the deflections of the vertical element increases and, consequently, a part of the water load is transferred into the horizontal arches. The part of the water load thus transferred increases until the elevation is reached at which the deflections of the vertical element are of such magnitude that the entire water load is carried by the horizontal arch element. At still higher elevations a reversal of conditions takes place. The vertical element does not take care of any water load; on the contrary, it increases the loadings of the horizontal arch

elements. At these elevations the loading of the horizontal arch is greater than the corresponding water pressure and the vertical element is loaded in the up-stream direction.

In general, the action of each vertical element is such as to assume loadings that will modify the loading of the horizontal arch element, making it zero at the points of greatest hydrostatic pressure and increasing it toward the points of zero hydrostatic pressure.

Due to these loadings the vertical crown element of a symmetrical dam has certain horizontal and vertical deflections, that appear before and after the water load is applied, as follows:

(1) Vertical deflection due to the weight of the dam without water load;

(2) Horizontal radial deflection due to the weight of the dam without water load;

(3) Horizontal radial deflections due to the moments introduced by water load; and,

(4) Horizontal radial deflections due to the shear introduced by water load.

Deflection (1) can be neglected as far as the arch dam design is concerned, and only Deflections (2), (3), and (4), need be considered. The total deflection of the vertical element that enters into the design is the difference between the horizontal, radial deflections at water load and under no-load conditions.

The deflections of a vertical element anywhere between the crown and abutment, or the deflections of the crown element of an unsymmetrical dam, are of the same character under no-water load, but are much more complicated under water load.

The vertical crown element of a symmetrical dam is symmetrical and considering a part of such an element cut out by horizontal planes the resultants of the water pressure carried by the vertical element are applied in the same vertical plane. All other vertical elements of a dam are unsymmetrical and the resultants are applied at different planes. Fig. 9 illustrates the lower part of such an element. The radial load at each elevation is different in direction, as well as in magnitude, and, consequently, the loads are not in a vertical plane through the center of the base.

Due to the fact that all the forces are not parallel, the deflection of the element will consist of a radial deflection, tangential deflection, and torsion, as indicated by the dotted lines, which show the deflected position of the element, greatly exaggerated for clearness. The radial and tangential movements of the center of gravity, G, of the horizontal plane at the top of the element, to the position, G', is shown, and the rotation of the axes through this point is also indicated. As a result, instead of the horizontal radial deflections of the symmetrical vertical element, deflections are introduced that are not radial. Considering the deflections of the centers of gravity of each horizontal section, the total can be resolved into radial, tangential, and torsional deflections, in a horizontal plane.

The proper procedure in determining the total deflection of a vertical section from the condition of no load to full water load would be to compute the horizontal deflections under water load due to the combined dead weight

and water load and to deduct (with the proper sign) the deflections due to the dead weight only. If tension of any magnitude is developed in the horizontal joints of any of the vertical elements, then the deflections cannot be computed in the conventional way and the behavior of the horizontal joints must be considered.

In computing the deflections of a vertical element it is of importance to include the influence of the elastic deformations of the foundation rock on which the element rests, this deformation resulting in an increase of the horizontal

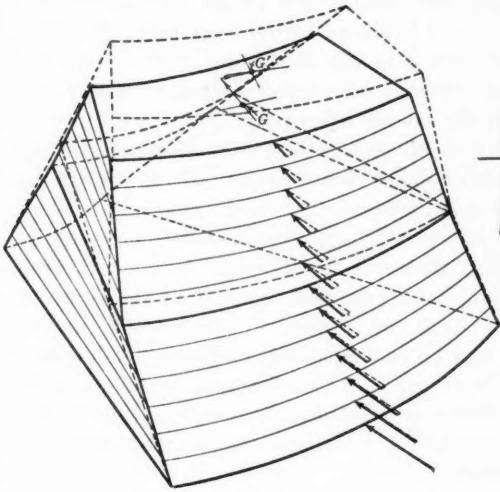


FIG. 9.

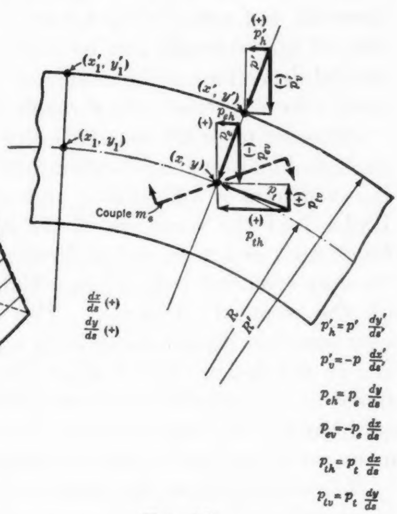


FIG. 10.

deformations due to the moment and horizontal shear applied at the base. The deflection of the vertical dam elements is influenced considerably by the foundation deformation. This increased deflection becomes more and more important with increase in height of the dam.

If the design does not take care of these conditions cracks in the concrete or the openings in the horizontal joints at one of the faces of the dam are inevitable.

VII.—Continuity of the Structure

The investigation thus far has been based on the assumption of independent arches and independent vertical elements, so that the deflections of each point of a horizontal arch or vertical element were unrestrained. In order to transfer each of these independent arches and elements to an actual structure, a number of additional restrictions must be imposed upon the independent arches and elements, which will cause the dam to act as a unit, thereby retaining the continuity of the structure.

The dam can be considered as made up of a number of voussoirs that are formed by parallel horizontal planes and radial vertical planes. Each voussoir is a part of a horizontal arch, and, at the same time, it is a part of a vertical element.

Considering the horizontal arches and the vertical elements separately, the conception would be that the dam consists of a number of arch voussoirs and an equal number of voussoirs of the vertical element, that each voussoir of the horizontal arch is identical with a corresponding voussoir of a vertical element, and that these voussoirs coincide.

The continuity of the structure under load requires that the corresponding voussoirs that coincided before the load was applied, must coincide afterward. Due to the load application the horizontal arches and the vertical elements will deflect. Disregarding the small deflections in the vertical direction and considering deflections in horizontal directions only, the deflection of any voussoir can be represented as the horizontal, radial, and tangential deflections of its center of gravity and a torsion of the voussoir with respect to a vertical axis through the center of gravity.

In order to fulfill the continuity requirements the torsional deflections and the radial and tangential deflections of the center of gravity of the corresponding voussoirs of a horizontal arch and the vertical element must be equalized. Under load the voussoirs of the horizontal arch elements will have radial, tangential, and torsional deflections that will be different, in general, from the corresponding radial, tangential, and torsional deflections of the voussoirs of the vertical elements. This lack of coincident deflections in the voussoirs that should coincide, is a proof that all the conditions which should govern the design are not given the proper consideration and that some additional forces and moments were neglected. To make a complete design these additional forces and moments should be superimposed upon the forces and moments that were determined originally.

The continuity of the structure is a natural condition that will be assured irrespective of the design assumptions, and the additional conditions that must be considered can be found approximately by evaluating the additional forces and moments that are necessary to equalize the deflections of the coinciding voussoirs. These forces and moments must be applied simultaneously at the voussoirs of the horizontal arches and vertical elements, and they must be in equilibrium as far as the entire structure is concerned. They will represent forces and moment couples transmitted from the voussoirs of the horizontal arches to the voussoirs of the vertical elements and *vice versa*.

These moment couples and forces will disturb the compensation of the horizontal arches and a re-adjustment must be made to re-establish the conditions of compensation. Such re-adjustment can be made by trial, computing the moment couples and forces that are to be applied at the voussoirs of the vertical elements to effect the necessary additional displacements and torsions to bring them into the proper position. These moment couples and forces must be applied in opposite directions on the corresponding voussoirs of the horizontal arches, and as far as the arches are concerned they can be considered as external forces and moments.

These external forces and moments can be substituted by radial and tangential forces and moment couples that are applied at the centers of the arch voussoirs. The moment couples acting along the center line of the arch

can be substituted by radial forces. If the moments are designated by m_e and the radial forces by p_e, then,

$$p_e = \frac{d\,m_e}{d\,s} \qquad \text{.........................(86)}$$

The horizontal arch will finally be loaded by the radial forces, p', due to water load, and by the radial forces, p_e, and tangential forces, p_t, due to the action of the vertical element.

In order to determine the conditions necessary to retain the continuity of the structure, a procedure similar to that presented in Section III may be followed, introducing the radial forces, p_e, and the tangential forces, p_t, acting at the center line of the arch, together with the radial loads, p', acting on the up-stream face. These loads may be resolved, as follows, into their vertical and horizontal components (keeping in mind the positive and negative values of x, y, s, p', p_e, and p_t as indicated in Fig. 10):

$$p'_h = p' \frac{d\,y'}{d\,s'}\,;\ p_{eh} = p_e \frac{d\,y}{d\,s}\ ;\ p_{th} = p_t \frac{d\,x}{d\,s}$$

$$p'_v = p' \frac{d\,x'}{d\,s'}\,;\ p_{ev} = -\ p_e \frac{d\,x}{d\,s}\,;\ p_{tv} = p_t \frac{d\,y}{d\,s}$$

The new equations derived by using the foregoing loads on the arch are:

$$\frac{d\,t_1}{d\,s_1} = 0 \qquad \text{.........................(87)}$$

which is identical to Equation (28);

$$d\,T_1 = p_{t1}\,d\,s_1 \qquad \text{.........................(88)}$$

which corresponds with Equation (34); and,

$$T_1 = p'_1\,R'_1 + p_{e1}\,R_1 \qquad \text{.....................(89)}$$

which corresponds with Equation (37). Integrating Equation (88),

$$T_1 = \int_0^s p_t\,d\,s + T_{aR} \qquad \text{.....................(90)}$$

or $T_{aR} = T_1 - \int_0^s p_t\,d\,s$, in which, T_{aR} is the thrust at the right abutment.

Re-arranging Equation (89), and substituting Equation (4),

$$R_1 = \frac{T_1}{p_1 + p_{e1}} \qquad \text{.......................(91)}$$

Equation (90) shows that in order to have uniform thrust the tangential forces, p_t, applied at the center line of the arch must be made equal to zero and then from Equations (91) and (90), $R_1 = \dfrac{T_{aR}}{p_1 + p_{e1}}$, or, substituting Equation (86), $R_1 = \dfrac{T_{aR}}{p_1 + \dfrac{d\,m_{e1}}{d\,s_1}}$.

In a trial design if the radial deflections of the vertical elements are larger than the corresponding deflections of the arch blocks, the continuity of the structure may be satisfied by an increase in the size of the **V**-shaped vertical joints and a resulting increase in the deflections of the arch blocks. The vertical stresses on the down-stream face of the vertical element should be adjusted by this means in order to avoid or reduce tension and to reduce the avoidable overloading of the upper arches.

The steps by which a compensated dam is designed, taking into consideration the restrictions that the continuity of the structure impose upon the independent horizontal arches and independent vertical elements, may be summarized as follows:

1.—The radial deflections of the centers of gravity of the corresponding voussoirs of the independent horizontal arches and independent vertical elements are equalized by a properly assumed distribution of the water loads between the horizontal arches and the vertical elements.

2.—The tangential deflections of the centers of gravity of the corresponding voussoirs of the horizontal arches and vertical elements are equalized so as to eliminate the tangential force, p_t. This condition will govern the relative shapes of the dam at different horizontal elevations.

3.—The radii of the horizontal arches are varied so as to take care of the twisting moments, m_e, as well as the assumed part of the water load. The moments, m_e, are introduced in order to equalize the torsional deflections of the corresponding voussoirs.

4.—A trial design is made to determine the increase in the deflection of the horizontal arches that may be necessary to improve the conditions of the vertical elements.

Steps 1 and 3 must be followed in any case, but it is not necessary to follow Step 2 in order to have a compensated dam with zero bending moments at the horizontal arches. If this condition is not met, then a compensated dam will result with varying thrusts and stresses along the horizontal arches.

VIII.—Straight-Line Distribution of Stresses

In connection with these analyses it is important to visualize the errors due to the assumption of straight-line distribution of stress. This arbitrary assumption is commonly made in considering the vertical elements of a dam as well as the horizontal arches.

At the points where the vertical elements meet the foundations, or where the horizontal arches meet the abutments or the foundation rock, the distribution of stress is radically different from the assumed straight-line distribution. A considerable increase of stress toward the surface occurs with a corresponding decrease toward the middle of the section.

The only manner in which the stress can approach, at least approximately, the straight-line distribution is to avoid sudden changes from one section to another at the abutments, or from a section of a definite thickness to the indefinitely large foundations, by using fillets to transfer the stresses from

the vertical elements or from the horizontal arches gradually to the foundations or abutments. Such details would seem necessary not only to have a better agreement between the assumed and actual conditions, but to avoid the high surface stresses at the juncture of the dam proper and the foundation rock or abutments.

IX.—Influences of Temperature, Expansion, and Contraction of Concrete, and Displacements of the Foundations and Abutments

During construction, the temperature, due to chemical processes still going on in the concrete, will usually be much higher than the mean temperature that the dam will attain in service. Before the arch dam is closed, the changes in temperature of the separated piers or concrete blocks is of little concern in so far as they affect the design; but every time the horizontal arch is closed at any elevation, arch action is made possible at this elevation, and the changes in temperature will influence the stresses. From this point of view, the concrete temperature of a horizontal element at the time the last of the alternate blocks or piers is filled in, should be considered as the starting temperature to which all subsequent temperatures are to be referred. The temperatures at this time may be quite different in different blocks and at different points, but an average closing temperature at each elevation may be determined for the design. Due to the difference between the temperature of the dam in service as compard with the closing temperature, a certain re-adjustment of deflections and stresses must take place.

Considering a compensated arch dam with radii adjusted to the loadings, the change of temperature will change the deflections of the horizontal element. For a rise in temperature the horizontal element will deflect in the up-stream direction, and, consequently, the load transmitted to the horizontal arch element will increase, and the loading of the vertical element will decrease at the lower parts and increase at the higher parts of the dam. At the same time the horizontal element will become unbalanced (since the product of the new loading and of the up-stream radius of curvature becomes variable), and certain bending moments will be introduced in the horizontal arch due to external loadings.

The rise in temperature will increase the length of the horizontal arch and will introduce bending moments that oppose the moments due to rib-shortening.

Considering the horizontal arch under a rise in temperature the following counteracting influences will occur:

1.—The length of the arch will increase, which will increase the loading of the horizontal element; the increase of the loading, in turn, will increase the rib-shortening and will partly compensate the increase of the length due to rising temperature.

2.—The rising temperature will introduce bending moments that will be partly compensated by the bending moments due to the unbalancing of the product of the load times the up-stream radius.

Under falling temperature a reversal of these conditions will take place. Similar conditions will be encountered if volumetric changes occur in the concrete, as, for instance, in the swelling due to the so-called water-soaking effect.

In these and in similar cases it can be seen that the properly designed and compensated arch possesses a very remarkable self-adjusting feature, which results in a more uniform redistribution of stresses that are imposed upon the dam.

However, there is one important case for which the arch dam cannot be designed to improve conditions and must take all the "punishment," namely, when there is elastic deformation in the rock foundations. Such deformation has an important bearing on the stresses developed in the dam and, consequently, on the design.

In most structures these deformations have little effect on the design as long as they are proportional to the loadings, but in an arch dam their importance cannot be over-emphasized and the lack of reliable information on this subject is a considerable handicap. The assumption made so often, that the foundation rock deformations are negligible, does not conform with actual conditions, and results in inadequate designs.

This case stresses still more the importance of properly distributed **V**-shaped joints, relieving the arch from stresses that can be avoided.

X.—Symmetrical and Unsymmetrical Dam Sites

In the application of this analysis to a symmetrical site a symmetrical arch dam will result; it will have a constant thickness at any given elevation and its radii will vary at different points at that elevation, being adjusted to correspond to the distribution of the loadings between the vertical and horizontal elements. The shape of the dam will relieve the horizontal arches from bending moments due to external loading, and the **V**-shaped vertical construction joints will relieve them from moments due to rib-shortening, difference in temperature, and displacements at the foundations and abutments. At the same time, the thrust and stress in the horizontal direction will be constant at each elevation.

By increasing the ratio of the radius of curvature to the span of the dam, the load sustained by the vertical element will increase until, for a ratio equal to infinity, all of it will be carried by the vertical element and the arch dam will be transformed into a straight gravity section. By decreasing this ratio the part of the loading carried by the horizontal arch elements will increase, the influence of gravity action will decrease, and the dam will approach a circular shape. The theoretical limit will be reached when all load is transmitted to the horizontal arch.

For an unsymmetrical gorge all the main characteristics of a compensated dam—constant thickness, constant product of horizontal arch loading and up-stream radius of curvature, constant thrust, and stress at each elevation—remain the same. Applying the same reasoning that was used for a symmetrical site to an unsymmetrical gorge, it will be found that an unsymmetrical arch dam will result, having the same properties as the symmetrical

dam for a symmetrical canyon. In general, this analysis covers any kind of symmetrical and unsymmetrical dams within the limits of the gravity dam and the straight cylindrical segment. The same method of design can be applied to curved gravity dams.

As far as theory is concerned, no difference exists between an arch dam and a curved gravity dam, and both should be designed in the same way. When the radii of curvature of an arch dam are increased under the same maximum stress assumptions, the thickness of the vertical elements will increase and the stresses in the horizontal arches will decrease. Practically that would mean that the load at the sides of the gorge or at the abutments is decreased and the load at the bottom of the canyon or at the foundations is increased, until for a straight dam the sides of the canyon or the abutments are relieved from any load and the total load is taken care of by the foundations.

From a purely theoretical point of view, any desirable distribution of loads between the foundations and the sides or abutments is possible. For the straight gravity section the sides of the gorge are relieved from load, but the vertical section is heavy, and high stresses are developed in the foundations. By loading the sides of the gorge, the vertical section can be made lighter and the stresses can be reduced at the foundation.

For the gravity dam the stress assumptions of the vertical element at no-water load and at water load will govern the shape and size of the vertical cross-section, and practically only one design that meets these assumptions is possible. For a compensated arch dam a number of designs is possible which will meet the stress specification of the vertical element, but will differ as to the curvature of the horizontal elements and the size of the vertical cross-sections.

If a heavy vertical crown section approaching the gravity section is assumed, an arch dam with a small degree of curvature and with low horizontal stresses in the arches will result.

If a lighter vertical crown section is assumed, an arch dam of increased curvature and with higher horizontal stresses in the arches will result. The most economical design as far as the quantity of material is concerned, will depend on the shape of the dam site. For narrow, deep canyons such design will require a dam of relatively thin vertical cross-section with considerable curvature; for wider gorges it will approach more closely the gravity dam, with less curvature and a heavier vertical cross-section. The most economical design for each dam site can be determined by the comparison of a number of trial designs with differently assumed crown sections.

In choosing the type of dam for a given site, it is generally conceded that arch dams are economical only for narrow gorges, say, less than 1 000 ft. wide. Actually, the economy depends more on the height of the dam than on the width, and the limits of width that justify the use of arch dams increase rapidly with the increase of the height. For a dam more than 200 ft. high an arch dam probably would be more economical than a gravity dam for much wider gorges, provided the sides can withstand the loads safely without undue deflections.

XI.—Outlines of the Proposed Method of Design

An attempt to apply the results of this theoretical investigation to the design of an arch dam for a given dam site will show that, due to the large number of variables, it is rather difficult to solve the problem in the customary way. The solution is more feasible if the procedure is reversed, by making a number of assumptions concerning the cross-section of the dam, the loading distribution, and the arch deflection. The theoretical dam site that fits these assumptions should be found and a number of trials should be made under different assumptions until one of the theoretical sites will fit the actual dam site. The steps to be followed in these trials are as follows:

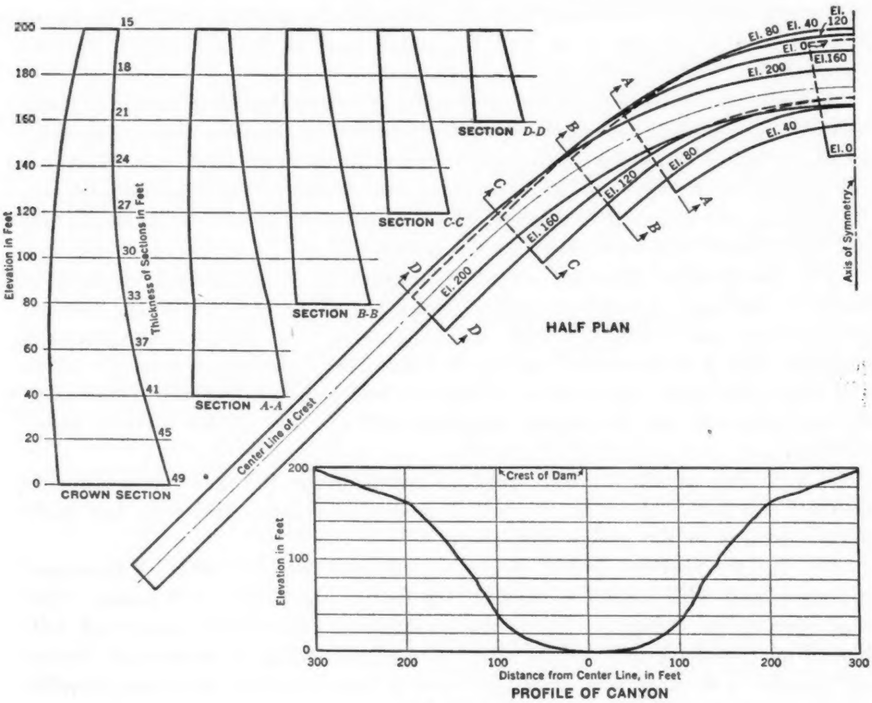


FIG. 11.

1.—The vertical crown section and its loading curve are assumed and the deflection curve is drawn. The loading curve of the horizontal arch at the top of the dam and the length of the top arch must also be assumed.

2.—For the first attempt it is convenient to assume the radial deflections of the horizontal arches to be proportional to the loads sustained by these arches.

3.—Beginning at the top, determine the radius of curvature at the crown, which will give a radial deflection of the horizontal arch at the crown equal to the radial deflection of the corresponding point of the vertical crown section. By trial, determine the radius of curvature at the next lower hori-

zontal arch which will have at each voussoir the same relative radial deflection with respect to the upper arch, as the vertical elements have with respect to the corresponding voussoirs. This procedure is continued for each successive lower elevation and the shape of the dam site is found that corresponds to the assumed set of conditions. Fig. 11 shows such a preliminary layout of an arch dam, with the shape of the resulting dam site.

4.—The tangential and torsional deflections of a few points in different horizontal arches are determined.

5.—The tangential and torsional deflections are computed at a number of elevations for a few vertical elements and these are compared with the corresponding deflections of the horizontal arches. The necessary adjustments are made in the shape of the dam and of the dam site that will equalize the tangential deflections, at the same time revising the radial deflections and the radii of curvature to take care of the additional moments introduced in the horizontal arches.

6.—This preliminary design is modified to take care of any secondary influences that the designer cares to be included.

7.—After the final design is made, the sizes of the **V**-shaped joints are computed, taking into consideration the rib-shortening, abutment deflections, difference in temperatures, and the necessity (if any) of increasing the deflections of the horizontal arches.

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PAPERS

DEVELOPMENT OF HYDRO-ELECTRIC POWER AS AN AID TO IRRIGATION[1]

By Charles C. Cragin,[2] M. Am. Soc. C. E.

Synopsis

Fundamentally, the economic feasibility of hydro-electric development in connection with an irrigation project presents the same problems for consideration and is governed by the same limiting factors as any other hydro-electric development, with the exception that certain additional restricting considerations are introduced in the case of the combined development.

This paper discusses these restrictions and propounds three fundamental principles to govern the engineer in judging the feasibility of combination. Eight basic essentials for a successful hydro-electric power system, and the fields most appropriate for the use of power in connection with an irrigation project are presented. Supporting illustrations are drawn from the Salt River Project in Arizona. Six conclusions are listed at the end of the paper.

In an irrigation project, the reclamation of large areas of land and the farmer's investment in land, buildings, and other improvements is dependent entirely on the availability of an assured water supply. The availability of an acre-foot of water, or the lack of it, may mean success to the farmer's agricultural operations on the one hand, or failure, with possible loss of his entire investment, on the other. The loss due to the lack of an acre-foot of water for irrigation may be anything from a few hundred dollars to several thousand dollars; whereas the loss of revenue to a power system would be relatively small.

It is conceivable that in a project which includes a power development in connection with irrigation storage a situation might arise such that in the face of an insistent demand for power and a concurrent need for money, there would be a strong temptation to waste stored water in order to realize immediate cash revenue. This is a practice which might be attended with serious agricultural losses due to resulting shortage of irrigation water at a critical

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[2] Gen. Mgr. and Chf. Engr., Salt River Val. Water Users' Assoc., Phoenix, Ariz.

time; and it might defeat the main purpose of the project. Such serious shortage of irrigation water might affect the power market even more than the loss of the same quantity of water for the production of power. The following fundamental principle therefore must be accepted, namely:

A.—Power development in connection with an irrigation project is justified only when there is a little or no interference with the irrigation system.

This is an economic fact; and, undoubtedly, it is the basis of the laws concerning this subject in most of the Western States.

The term, "irrigation project," is ordinarily understood to mean an engineering development having for its purpose the irrigation of otherwise arid land for agricultural purposes. The property involved is usually owned by a large number of individuals whose main interests and training have to do with farming. A power project is ordinarily proposed and constructed as an investment of established capital, usually, in the more modern installations, for the purpose of re-selling the power produced to the general public for all purposes ranging from small domestic uses up to large industrial uses. The operating organization of a purely irrigation enterprise is rarely adapted to the production and marketing of such a commodity as electrical energy. This is a highly specialized industry and requires the services of a force of specialists entirely foreign to the activities encountered in an irrigation enterprise. To a public utility or a power company operating independently on a fairly large scale (in which case there would be ample assurance of an adequate stand-by service), a development which might reasonably be expected to show a profit of from 8 to 10% on the investment, would be considered a favorable undertaking. The same physical conditions and the same development made thereunder by an irrigation project might be a very doubtful investment. A much larger profit on the increment investment should be the minimum set for the same development by the irrigation project.

Most irrigation enterprises are developed to the maximum extent economically feasible at the moment, with or without provision for future expansion, depending on local conditions. Financing is frequently difficult and the expenditure required per acre of land benefited sometimes approaches the allowable limit. To saddle such an enterprise with the risk of having to develop power during water shortage, or other periods of stresses, such as to become a burden to land already heavily taxed, might easily place the financial affairs of the entire irrigation enterprise in serious jeopardy and even cause many individuals to lose their entire investment.

The power business constantly requires large sums of money to meet the ordinary growth of a community and the increased uses of power, to keep step with invention and improvement in the electrical industry, and to meet the standards of better service set by the public utilities of the country. Mobility and ability to develop speed in financing are essential; but these are almost always absent in irrigation projects, practically all of which require votes of their members or stockholders for all substantial financing. One large power development in connection with irrigation requires a three to one majority for authorizing any expenditure greater than $100 000 in

any year. Bond issues of this nature are always subject to legal contest by any portion of the minority. Known examples of six months' delay have been caused from litigation carried on by less than 1% of the stockholders. The Salt River Project in Arizona develops more than 40% of all power produced by irrigation projects. This project, on account of favorable physical market condition, has had a wide margin between gross and operating revenue. This has made possible a concession in rates in return for the advance in cash of more than \$2 500 000 from its customers toward the necessary investment. In addition, other rate concessions have been made in return for minimum guaranties to purchase as much as \$40 per kw-year for wholesale blocks of 7 000 kw. and up. More than \$9 000 000 worth of bonds out of the total outstanding of \$14 000 000 on the project were sold advantageously by pledging these guaranties, in addition to liens on the project.

Another item to be considered in connection with the necessity of a wide margin of profit is the effect on the credit of individual farms. The writer's experience leads him to believe that one of the most important causes of agriculture's trouble to-day is lack of proper financing of the farm. A reasonably profitable power development in connection with an irrigation development, which might affect disadvantageously the personal credit of an individual member of such project would be a most uncertain benefit. A relatively small curtailment of this individual's credit would more than wipe out the share of the reasonable profits due him from the power system as a member; hence, the following principle:

B.—A power development, therefore, made in connection with an irrigation development, may be considered advisable only when the margin of profit assured is greatly in excess of that required in an independent power development.

It does not follow, however, that a feasible power installation in connection with an irrigation project must remain undeveloped because the margin of profit is too small to justify the undertaking as a part of the irrigation system. In projects with only a reasonable margin of profit, the element of risk may often be shifted to other shoulders by leasing or selling the power privileges under appropriate contract which will assure freedom from interference with the water supply or operation of the irrigation system, and provide power for project needs under favorable terms. If the physical and hydrological conditions, taken in connection with the power demand, are not sufficiently favorable to make such a lease or sale readily possible to a responsible public utility, power company, or industry, it may be taken for granted that the development is one which the irrigation project should under no circumstances undertake. In the event, however, that the element of risk is substantially eliminated, and it appears advisable that the development be made by the irrigation project itself, it would still be good business for the output of such a power plant to be disposed of in one or more large blocks rather than for the irrigation project to engage in the general retail distribution of power. In other words:

C.—Only under the most unusual circumstances will an irrigation project be justified in engaging in the general power business.

This general statement, as made, is presented as a third fundamental factor of almost equal importance with the two principles before enunciated, differentiating between a hydro-electric development in connection with irrigation and a hydro-electric development made independently.

One advantage often possessed by a power system constructed in connection with an irrigation project, which is usually not possessed by a development made for power alone, is that irrigation developments of sufficient magnitude to contemplate the installation of a power system on any considerable scale are generally organized as irrigation districts under State laws or are Federal Reclamation projects. In either case, the project is exempt from taxation, and in addition enjoys whatever benefit there may be in freedom from control by State regulating bodies; but such freedom from taxation should not be considered wholly as profit. As far as such a project is concerned it is obvious that the profit from this tax-free circumstance is limited by the amount of the lost taxes which are made up by tax-payers other than the project members. The benefit from this would be the elimination of that amount of financial risk represented by the amount of the annual taxes thus saved, and not by the full amount of the taxes.

The basic essentials for a successful hydro-electric power system are too elementary and well-known to require elaboration. Briefly, these essentials are, as follows:

1.—Physical and hydrological conditions must be favorable.
2.—Market must be assured.
3.—Hydro-electric costs must compare favorably with steam costs.
4.—There should be freedom from competition.
5.—The market must be within economical transmission distance.
6.—The load must be adapted to the production capacity.
7.—Financing must be economically feasible.
8.—Design and construction of works must be of high character.

These and other factors apply to any power development. A hydro-electric development made in connection with an irrigation project, however, as before stated, would be considered feasible only to the extent that the irrigation system is not interfered with and in cases where the margin of profit is much larger than in an independent development. In view of the great number of irrigation storage dams existing and proposed in the United States and in other countries, it might be expected that the opportunities afforded for hydro-electric development in connection with such storage dams would be very numerous. The truth, however, falls short of this to a rather surprising degree. Of the twenty-eight irrigation projects of the United States Bureau of Reclamation operating on December 31, 1931, on which there are forty-two storage dams, hydro-electric power has been developed on only twelve. On three of these projects, the developments are of small capacity only—less than 200 kv-a.—so that there are actually only nine commercial developments. Of the remaining plants, the largest installations are the Salt River Project, 83 210 kv-a.; Boise, 11 875 kv-a.; Minidoka, 11 540 kv-a.; North Platte, 7 750 kv-a.; Riverton, 1 000 kv-a.; Shoshone, 2 000 kv-a.; Strawberry Valley, 1 000 kv-a.; and Yuma, 2 000 kv-a. The gross capacity of all plants is

122 960 kv-a., of which more than two-thirds is on the Salt River Project. The output for the year ending June 30, 1929, for the entire twelve projects was 345 300 000 kw-hr., of which 206 000 000 was produced by the Salt River Project, without the new Stewart Mountain development, finished in March, 1930. The foregoing figures are quoted from the report of the Commissioner of Reclamation for the fiscal year ending June 30, 1929.

Power plant capacity of installations other than the Salt River Project as of June 30, 1931, was 39 977 kv-a., which produced (including steam power output), 160 796 586 kw-hr., totaling gross sales of $773 436.27. The power plants of the Salt River Project, however, of this date, total 83 210 kv-a., and produced (including steam power), 294 464 301 kw-hr., or a total gross sales of $2 686 301.72.

Almost every condition that can arise in connection with a proposed hydro-electric development exists on the Salt River Project. The project operates eight hydro-electric plants with a combined generating capacity of 103 000 h.p., including the one at Stewart Mountain Dam. These plants are located at the Roosevelt, Horse Mesa, Mormon Flat, and Stewart Mountain Dams, on Salt River, and at drops on the canal system in the Salt River Valley. Horse Mesa, Mormon Flat, and Stewart Mountain Dams and plants have been built in the six years since 1924, the Stewart Mountain Dam having been completed in March, 1930. The Salt River Project is primarily an irrigation enterprise and serves water directly to 250 000 acres of project land, and to 90 000 acres of non-project land. By project land is meant that which is entitled to water from Roosevelt, Horse Mesa, Mormon Flat, and Stewart Mountain Reservoirs and signed up in the Salt River Valley Water Users' Association, the organization of 9 500 farmers who own the land and operate the project. The non-project land served, consists of various areas receiving pumped water from the project; these developments owe their existence largely to the availability of hydro-electric power from the Association's plants. Owing to the number and variety of power installations on the Salt River Project, and the fact that nearly all factors to be considered in any power development connected with irrigation will be found on this project, it will be used to illustrate points to be brought out in this paper.

Use of Power, with Illustrations from the Salt River Project, Arizona

The fields most appropriate for the use of hydro-electric power developed in connection with an irrigation project, are the supplying of power in large blocks: (1) For irrigation pumping; (2) for mining and other industrial uses; and (3) to public utilities distributing and serving power at retail for general domestic and kindred purposes. Ordinarily, the organization and functions of an irrigation project are not adapted to the distribution of power at retail for domestic consumption. This statement is meant to apply to conditions in general, but like any general statement it is subject to modification when applied to a particular problem. In this case, an exception might well be made as to the delivery of power for domestic and farm use to the land owners

in an irrigation project that owns a hydro-electric power system capable of serving them without thereby entering into destructive competition with an existing utility.

Many isolated cases will also be found in which service cannot be obtained from other systems, but in which such service is feasible from the lines of an irrigation project. In any event, the service of electricity for retail consumption should be undertaken only in connection with an irrigation project in a new territory not already served from any other source. Where such new territory adjoins or overlaps territory served from other sources, competition and duplication of service lines and other installations should be avoided by agreement with other organizations serving power in that field.

Irrigation Pumping — The Ideal Load

The ideal load for a hydro-electric system, as part of an irrigation project, would ordinarily be an irrigation pumping load. Preferably, as before stated, this should be on the irrigation project that includes the power system, or on adjacent lands where the conditions of time and quantity of use are identical or closely similar. The use of power for drainage pumping would come under the same head, but would have the additional advantage of allowable latitude in time of operating the drainage pumps, so as to make possible the use of off-peak power.

In the design of a hydro-electric plant appurtenant to an irrigation system, the known irrigation pumping and drainage loads that may be available to increase the load factor of the power system as a whole, may properly be considered in fixing the justifiable generating capacity to be installed. The fact that these loads are known will warrant the installation of generating capacity over and above the assured load for other uses by the amount of the pumping loads. The use of power for other project purposes of a permanent nature, such as the lighting and operation of project works, will often constitute a first demand on the project power system, but ordinarily, the sum total of such uses will be a small percentage of the entire output of the system.

Power for Public Utilities, Mines, and Large Industries

In the sale of power in large blocks to public utilities, mines, and industrial enterprises, the demand that can best be synchronized with the output of the system will be most desirable. When the customer owns a steam plant capable of supplying his needs or, at least, the deficiency in hydro-electric power, it is possible to co-ordinate the two sources of power so that the maximum load factor will be obtained from the hydro-electric system. Power for industrial or general domestic uses must be firm power and to the extent that there are daily and seasonal fluctuations in the uses of power which can not economically be met by the hydro-electric system, the difference must be made up by stand-by steam plants.

A power market is rarely created quickly—it is a growth. Such an existing market must necessarily have been supplied from some source (usually from steam), prior to the development of hydro-electric power designed to supply energy either in place of, or as a supplement to, the steam power.

Where this condition is found to exist, it may be taken advantage of in the hydro-electric development by designing the hydro-electric plant to care for the growth in the market above needs previously supplied from steam power and, in addition, to take the place of the steam power to a greater or less extent. The power rate must be sufficiently lower than the cost of generation by steam to warrant the substitution, this rate being at the same time sufficiently above the cost of hydro-electric generation to be profitable in the hydro-electric system. Where this is done in such a way as to obviate the necessity, which would otherwise have existed, of rebuilding, replacing, or modernizing an existing steam plant, the two may be co-ordinated to particular advantage, since, in this case, the steam plant may still be retained as an auxiliary stand-by while the hydro-electric system becomes the main source of supply.

An example of this exists on the Salt River Project. The financing of the Horse Mesa Dam and its 44 000-h. p. plant was greatly facilitated by long-term power contracts with the Inspiration Consolidated Copper Company. This Company had a 40 000-h. p. steam plant, its maximum requirements for power being approximately 30 000 h. p. The steam plant was approaching obsolescence and it was, therefore, good business for the Mining Company to contract for the entire output of the Horse Mesa Plant, retaining its steam plant as an auxiliary and depending on Horse Mesa for its main supply. This steam plant, under the contract, in turn, becomes an auxiliary to the hydro-electric system of the Salt River Project, and is available to make up any shortage in power required at times for other customers. The only investment the Association was obliged to make for this 40 000-h. p. stand-by service was approximately $350 000 to bring the boiler capacity of the Inspiration Steam Plant up to the capacity of the generators.

A local condition on the Salt River Project unfavorable to steam, but favorable to hydro-electric plants, is the cost of fuel oil, the freight rate on fuel being $1.13 to $1.16 per bbl., or more than twice the price of the oil in 1930. This situation has changed temporarily, in that fuel can be contracted for short terms now (1932) at 50 cents per bbl., plus freight of about 87 cents. With this excessive fuel cost, steam power can not be generated for less than approximately 7 mills per kw-hr. in a modern plant; whereas, in the older type of plant mentioned, the cost of generation was nearer 1 cent. The increment cost of the hydro-electric power is approximately $3\frac{1}{2}$ mills. The contract price of $6\frac{1}{2}$ mills received by the Association for power delivered to the Mining Company is, therefore, a favorable one both to the Company and to the Association. The minimum annual payments guaranteed by this contract are more than sufficient to take care of the interest and principal on the development bonds of the Horse Mesa Dam, and the stand-by service makes firm a large block of Association power which otherwise would have to be sold as dump power.

Irrigation and Drainage Pumping Loads

The existence of an underground water supply susceptible of pumping for irrigation at economical cost, as before stated, may justify the installation of

generating facilities to furnish the power needed. The same is true of drainage pumping. Similarly, the availability of cheap power may warrant an irrigation pumping development which otherwise would not be feasible. The Salt River Project presents a notable example. In 1919, 23 000 acres of land were added to the 185 000 acres of the project, due to the development of pumps for utilizing the water from its large underground supply. The primary purpose of these pumps was drainage, but the installations were soon found to be an asset rather than a liability, due to the value of the water for irrigation, and the fact that operation for drainage could be effected largely by off-peak power. Beginning with 38 pumps in 1919, the installations by the Association were increased to a total of 206 at the end of 1930, the capacity ranging from a fraction of 1 cu. ft. per sec. to as much as 15 cu. ft. per sec. Twenty-six of these pumps, located at points where the water was of little value on Association lands, were sold to the Roosevelt Irrigation District, consisting of 40 000 acres adjacent to the Salt River Project on the west.

This District has installed 26 additional pumps within the Project and others within its own boundaries, and, in this manner, a dependable water supply has been developed. Power is furnished them from Salt River Project plants so that, not only is drainage provided for Project lands in the area where District pumps are operated, but a profitable outlet is provided for a substantial block of power. The total capacity of the 180 pumps operated by the Association is approximately 350 000 acre-ft. of water per irrigation season, if needed. During the abnormally dry years, 1920-31, this proved to be a valuable reserve to supplement the Association's irrigation supply from stored water and unregulated river flow. With high steam costs, this would not have been as economical as with with hydro-electric power and stand-by. The total project use of power for pumping in the year 1928–29 was 33 150 000 kw-hr., valued at $331 500.

The total water pumped by the Salt River Project was 259 000 acre-ft., of which only 26 000 acre-ft. was drainage water not susceptible of irrigation use, although a conservative estimate of the annual drainage pumping requirement of the project would be 200 000 acre-ft. The drainage benefits, therefore, may be considered as being almost entirely in addition to the value of the water for irrigation which, itself, is nearly $400 000. The Association also supplies power direct, for irrigation pumping on 20 000 acres of land west of Phoenix, and for pumping by the Roosevelt Water Conservation District. The latter is a 40 000-acre tract east of the Project, the water supply of which consists of the output from forty-seven wells and river water raised 50 ft., by pumps, from the main Southside Canal of the Salt River Project. Other irrigation enterprises supplied with power by the Project's hydro-electric system, include several electrical districts in the near-by Casa Grande Valley, and a number of isolated small irrigation developments. The irrigation pumping load of the Salt River Project, in addition to 33 150 000 kw-hr. used by its own pumps, amounted, in 1929, to 62 200 000 kw-hr. The aggregate pumping load on the Project's system for that year was, therefore, approximately 95 350 000 kw-hr., which equals the estimated combined output to be expected from the Mormon Flat Plant and the Stewart Mountain Plant.

Reclamation of Additional Land

A direct irrigation benefit on the Salt River Project from its power development is the increase of 10 000 acres in the Project area, made possible by the construction of the Mormon Flat Dam, which, itself, was purely a power development. This dam, however, impounds the run-off from approximately 300 sq. miles of the area below the Roosevelt Dam, and this stored water was thus made available for irrigation purposes.

The entire cost of this particular dam is charged to the power system, and in this case the irrigation benefit is entirely incidental. This project was not feasible if charged to irrigation alone, but the power feature made the development profitable.

Power for Construction

The Salt River Project furnishes an example of the use of power from its own hydro-electric system in the construction of both the irrigation and power features of the Project. The power used in the construction of the Roosevelt Dam, begun by the United States Reclamation Service in 1906 and completed in February, 1911, was furnished entirely from a temporary hydro-electric plant (later incorporated in the permanent plant). This was a 1 000-kw. installation, obtaining its head and water supply by means of a 200-cu. ft. per sec. power canal, 19 miles in length.

The power canal was designed to be a permanent feature of the hydro-electric system, being intended for use in times when the water level in the reservoir was low. In the later construction at Horse Mesa, Mormon Flat, and Stewart Mountain, all power was furnished from the Association system, being brought into the sites of the several dams over the same lines subsequently used to connect their respective outputs to the main transmission system.

Retail Delivery of Power for General Use

The Salt River Project also furnishes one of the instances illustrating conditions under which an irrigation enterprise may engage advantageously in the retail distribution and service of electric power for domestic and similar purposes.

There are 9 500 farms on the property embraced by the Project, a highly developed area, approximately 40 miles in length along its greatest dimension, and including 250 000 acres of irrigated farms and orchards. Practically one-half these farms are in close proximity to cities, towns, and other congested districts. In 1928–29, 700 miles of power lines were constructed in the valley of the Salt River Project, making possible the service of electricity to every farm. This development was made in connection with the construction of the Stewart Mountain Dam and power plant, a development which was financed after the making of a contract with a local public utility company, providing for the purchase of power from the plant and assuring minimum payments sufficient to take care of the indebtedness.

An important feature of the contract consisted of an agreement to avoid duplication of investment and a division of territory to be served from the

lines of the respective parties, under the approval of the State regulating body and the Secretary of the Interior. The Company serves 15% of the Project and the Association serves the other 85 per cent. By this agreement, the Association avoids any competition whatever with the existing public utility company, and the Company agrees to use the irrigation project's power, insuring a profitable market for 50 years. This has been a very successful development, and although the work of electrification was only completed in June, 1929, more than 80% of all homes on the Project had been connected by April 1, 1930. The revenue from this service in less than a year was sufficient to cover all additional costs, including interest and depreciation. The rapid building up of load was largely accomplished by establishing a profitable merchandising department. A well-arranged salesroom and force of ambitious and capable salesmen was found to be practically an essential. By this means not only was the number of connections brought to a profitable point in a short time, but the character of the load was greatly improved by the sale of many ranges, refrigerators, and other applicances.

Aid in Regulating the Running of Irrigation Water

An advantage gained from the power system on the Salt River Project, which may or may not find a parallel in other developments, is the closer regulation of the water discharged from the Roosevelt Reservoir, due to the construction of dams on the river nearer the point of diversion of the canal system. The Roosevelt Dam is 77 miles by road east of Phoenix, Ariz., approximately 45 miles above the Granite Reef (diversion) Dam at the head of the main canals serving the lands on both sides of the Salt River. It provides the main irrigation storage for the Salt River Project.

With the construction of the lowest dam at Stewart Mountain, the 45 miles have been reduced to 12 and practically true, demand-water service is available to the farmers. Practically any head of water, for any length of time, is delivered to the farm within 24 hours of the time for which it was ordered. This is 24 hours better than deliveries prior to the completion of the dam.

Development of Recreational Areas

A benefit not usually considered in connection with a strictly utilitarian development is the outstanding value for recreational purposes of the Horse Mesa, Mormon Flat, and Stewart Mountain Lakes. In a country essentially arid in character, any considerable body of water affords the element of novelty to an extent which greatly increases its popularity. These three dams create a continuous chain of lakes, 35 miles long, along the Salt River in the heart of spectacular rugged mountains, with cliffs rising at times 2 000 ft. sheer from the water's edge. The lakes extend to the Roosevelt Dam which, itself, creates a lake 25 miles in length.

The distance from Salt River Valley and the winding nature of the road has always greatly limited the use of Roosevelt Lake for recreational purposes. The lower lakes, however, are readily accessible. Stewart Mountain Lake in particular is only 40 miles distant, a matter of an hour's drive over an excel-

lent automobile highway from the City of Phoenix, with a population of 70 000. This recreational use, however, has its value from a strictly commercial standpoint, and it has been the incentive for the construction of high-class modern highways, for the advent of a large and growing winter population.

General Range of Benefits from Power

Using the Salt River Project as an illustration, it has been shown that benefits to be derived from hydro-electric power installations in connection with irrigation works may range all the way from the use of power for construction and other project purposes to the development of power as a commercial proposition independent of irrigation works, but providing incidental irrigation benefits in the form of better water service and permitting the reclamation of additional land. The conditions described, while peculiar to the Salt River Project, are not so unusual, but that they may be found singly or in combination in other kindred enterprises.

Development of Salt River Project Power System and Value in Money

The first power developed was purely for use in construction. With the completion of the permanent Roosevelt Plant, however, the capacity was made adequate to supply current in bulk to a local public utility company for general retail distribution. The original South Consolidated Power Plant and the Arizona Falls Power Plant were installed at drops on the main canals, with the Arizona Cross-Cut Power Plant (total, 10 000 h. p.), to meet the growing demand for power from the Project system. These developments were financed by direct assessments. The Chandler Power Plant, a 600-kw. installation made in 1919, was financed by the Magma Copper Company, under a contract whereby that Company agreed to take the entire output of the plant.

In 1920, the combined generating capacity of the Roosevelt Plant and the four valley plants was 20 000 h. p., and the investment of the Association in the power system was $4 500 000. The growth of the power demand in the meantime had been such that the Association was faced with the problem of installing additional generating capacity to take care of the market, or of leaving the field open for such installations by other competing interests. The Association's power business was undoubtedly profitable and its expansion, therefore, was simply a question of whether or not, as an irrigation enterprise, its officers were disposed to expand its power interests to any degree warranted by the market. An exhaustive investigation and study of all the conditions was completed and embraced in a comprehensive report in February, 1922.

The total yearly power consumption of the State of Arizona at that time was 500 000 000 kw-hr., of which 400 000 000 kw-hr. was within a radius of 100 miles of the Project plants; more than one-half these being within reach of existing transmission lines. Most of this load was generated by steam at costs approximating 1 cent per kw-hr. It was shown to be possible by the

installation of 15-ft. gates in the Roosevelt Spillways, and by the construction of three dams on the Salt River below the Roosevelt Dam, to make available a net head of 729 ft. out of a total of 832 ft. between the top of the gates at Roosevelt and the crest of Granite Reef (diversion) Dam. The increased generating capacity found to be economically feasible was 85 000 h. p., which was susceptible of immediate absorption by the then existing market.

The sites along the entire Salt River had been reserved for the benefit of the Salt River Project by the Secretary of the Interior in 1903, but it was obvious that in the event that the Association failed to avail itself of those sites and to develop their potentialities it could not reasonably be expected that they would not eventually be awarded to some one able and willing to develop them. This was the logical source of power to meet the local demand, and since the Association was already in the power business on a fairly large scale, the protection of its existing $4 500 000 investment made the further expansion of this activity a necessity, in addition to its advisability from the standpoint of a profitable undertaking.

The construction period extended over seven years, from 1923 to March, 1930. The increased storage facilities provided by the three dams in connection with the 15-ft. gates at Roosevelt, amounted to 648 000 acre-ft., bringing the gross storage on Salt River to 2 015 000 acre-ft. Although this additional storage was created solely for power purposes, and although the hydrographic records of Salt River indicated that its development would not have been warranted by the irrigation benefits alone; yet it is nevertheless true that this additional water would be available in unusual periods of drouth, if needed, to increase the irrigation supply for the project lands.

The main benefit of this development, however, is to be measured in actual dollars and cents. Disregarding the loss which might have been sustained had the Association failed to make this investment, leaving it to be developed by competing interests, it is estimated that once the reservoirs have been filled the gross annual power income will exceed $3 500 000, with a net profit of nearly $1 500 000, including power used for Project purposes. This income is expected to pay all Project indebtedness, all operation and maintenance costs, and to leave a substantial surplus besides. In comparison with the agricultural part of this enterprise, the net anticipated annual power income approximates $7.00 for each acre of land, which is a cash crop available whether or not the land itself is cultivated. During the eleven years of drouth (1920–1931) and during the period of development, the net profit averaged more than $600 000 per year, or $2.62 per acre per year. These profits and estimated future profits are based on the additional costs and investment incident to the production of power, and do not include any part of the original investment in Roosevelt Dam or the canal system, except the actual power structure and equipment.

In the last analysis, the benefits accruing to any irrigation enterprise from power developed in connection therewith could probably be reduced to terms of dollars and cents, even as the benefits from the irrigation development itself. In the case of the Salt River Project, the cost of works installed for irrigation purposes, and which would have been necessary therefor, with

or without the power, are charged entirely to irrigation, even if the power actually developed in connection with such works, such, for instance, as that at Roosevelt Dam, would have been entirely infeasible without the irrigation features of the installation. Similarly, the entire cost of works installed for power purposes, such as the Mormon Flat Dam, are charged to power even if an incidental irrigation benefit is derived, such as, in this case, the development of a water supply for 10 000 additional acres of land.

The actual investment on the Salt River Project for all Project works is approximately $29 000 000, although the replacement value would greatly exceed this amount. Considered as a power project alone, the revenues from this development, at 6%, would pay interest on an investment of $37 000 000. The actual amount of the investment charged to power, however, is $18 000 000.

Other Power Developments in Connection with Irrigation Projects

The San Carlos Project, under the Coolidge Dam, in Arizona, opened in 1929, presents physical features somewhat similar to those on the Salt River Project, and it is not impossible that its development may eventually follow along the lines found successful on the older one. The Coolidge Dam is situated at the head of a box canyon on the Gila River, not dissimilar to that of the Salt River. The river has a fall of nearly 500 ft. in the canyon below the dam and several power sites exist to make this development feasible.

Conclusions

The foregoing comments furnish arguments in favor of the following conclusions:

1.—The power development must not interfere with the irrigation project.

2.—The margin of profit should be much greater than in an ordinary hydro-electric development.

3.—If such a margin of profit is not available, power rights should be sold to produce power at the cost of steam or competing power.

4.—Only in rare instances, and under favorable conditions, should power be retailed by promoters of an irrigation project.

5.—Great benefit accrues to most projects from availability of power at low-increment cost for drainage and supplemental irrigation pumping, or both.

6.—A wide divergence exists between projects, some showing more benefits than others, but, in general, the benefits have overwhelmingly exceeded the drawbacks.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ANALYSIS OF CONTINUOUS FRAMES BY DISTRIBUTING FIXED-END MOMENTS

Discussion

BY J. A. VAN DEN BROEK, ASSOC. M. AM. SOC. C. E.

J. A. VAN DEN BROEK,[48] ASSOC. M. AM. SOC. C. E. (by letter).[48a]—Possibly the greatest single contribution to the theory of stress analysis of redundant structures in a generation is that presented by Professor Cross in this paper. Engineers have long been aware of the fact that the influence of a load on a support, several spans removed from the load, is negligible; yet, they have not found a legitimate way by which to avoid the cumbersome process of writing as many simultaneous equations as there are unknowns before attempting a solution. In a large number of problems commonly encountered, Professor Cross' method overcomes this objection. It appears to have been known and used for a number of years. When the subject was first called to the writer's attention, his immediate reaction was highly favorable. How simple! A veritable Columbus' egg! Upon closer study this first enthusiastic appreciation has grown rather than diminished.

Starting from the same premise, several variations in the detailed application of the method appear possible. In presenting one such variation on the same theme, which might possibly also be regarded as a variation on the slope-deflection theme, the writer is motivated solely by the thought that, because of the great importance he ascribes to the subject, the consideration of any modification in emphasis or point of view appears justified at this stage.

NOTE.—The paper by Hardy Cross, M. Am. Soc. C. E., was published in May, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1930, by Messrs. C. P. Vetter, L. E. Grinter, S. S. Gorman, A. A. Eremin, and E. F. Bruhn; October, 1930, by Messrs. A. H. Finlay, R. F. Lyman, Jr., R. A. Caughey, Orrin H. Pilkey, and I. Oesterblom; November, 1930, by Messrs. Edward J. Bednarski, S. N. Mitra, Robert A. Black, and H. E. Wessman; January, 1931, by Messrs. Jens Egede Nielsen, F. E. Richart, and William A. Oliver; February, 1931, by Messrs. R. R. Martel and Clyde T. Morris; March, 1931, by Francis P. Witmer, M. Am. Soc. C. E.; May, 1931, by Messrs. T. F. Hickerson, F. H. Constant, W. N. Downey and E. C. Hartmann; September, 1931, by Messrs. Thomas C. Shedd, David M. Wilson, and Marshall G. Findley; November, 1931, by Messrs. George E. Large, and Sophus Thompson and R. W. Cutler; January, 1932, by Alfred Gordon, Assoc. M. Am. Soc. C. E.; and March, 1932, by Messrs. A. W. Earl, A. Floris, I. M. Nelidov, E. A. MacLean, George M. Dillingham, and Donald E. Larson.

[48] Prof. of Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

[48a] Received by the Secretary February 1, 1932.

The basic principle in Professor Cross' method appears to be that first all joints are considered locked, and then released one joint at a time. As each joint is released all other joints are assumed as being locked. Each time a joint is unlocked the structure will deform toward the shape it will ultimately assume in its natural state. If all joints are successively unlocked

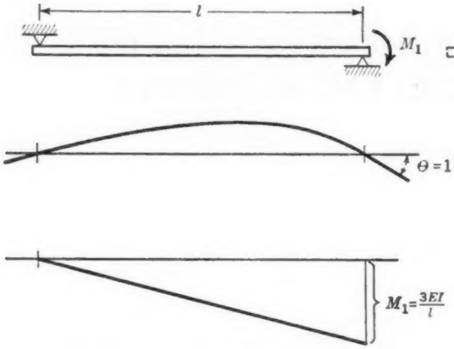


FIG. 66

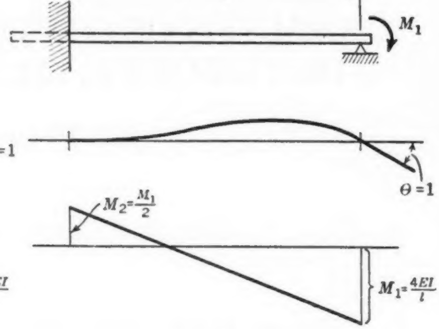
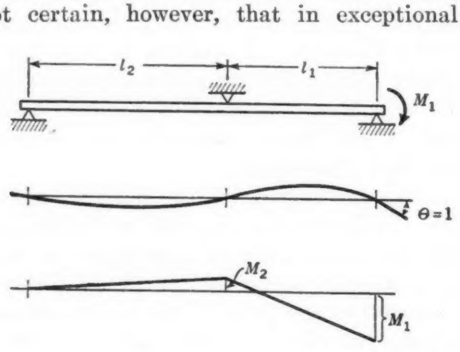


FIG. 67

and locked in turn, an infinite number of times, the structure will approach its final natural shape. Professor Cross appears to assume that this final shape will be approximated very closely after only two or three operations of locking and unlocking the joints. This premise appears reasonable. It is not certain, however, that in exceptional cases this convergence will be



$\frac{E_1 I_1}{l_1} = k_1$

$\frac{E_2 I_2}{l_2} = k_2$

$M_1 = \frac{12(k_1 k_2 + k_1^2)}{4k_1 + 3k_2}$

$M_2 = \frac{k_2 M_1}{2(k_1 + k_2)}$

When $k_1 = k_2$ then $M_1 = \frac{24EI}{7l}$; $M_2 = \frac{M_1}{4}$

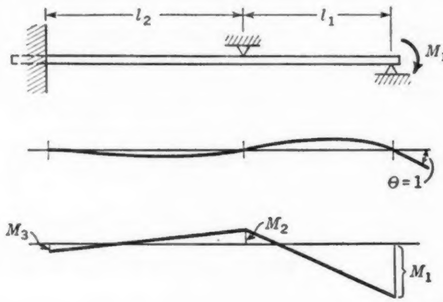
FIG. 68

achieved. A rigorous proof of the aforementioned premise, would be of much value at this stage.

Instead of conceiving joints successively locked and unlocked, one might conceive all joints except the pin joints as locked. Then proceed to unlock each joint, except those that are fixed, or built in. Note that in this method of approach the pin joints are never considered as locked and the built-in joints are never considered as unlocked. Furthermore, once any of the other joints are unlocked they are never to be considered as locked again.

Figs. 66 to 71 are to be regarded as a set of key diagrams to be used in connection with this method of analysis. In each case let M_1, the stiffness factor, represent the moment necessary to produce unit angular displacement. The elastic coefficient for any member is expressed by $k = \dfrac{EI}{l}$.

Let Fig. 72 represent a continuous beam of five equal spans, with E and I constant. The left end, Joint A, is fixed; the right end, Joint F, is freely



$$M_1 = \frac{4k_1k_2 + 3k_1^2}{k_1 + k_2}$$

$$M_2 = \frac{2k_2 M_1}{3k_1 + 4k_2}$$

$$M_3 = \frac{M_2}{2}$$

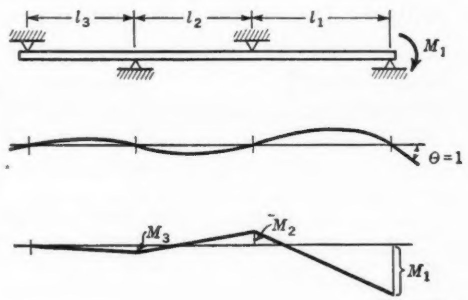
When $k_1 = k_2$ then $M_1 = \dfrac{7EI}{2l}$;

$$M_2 = \frac{2}{7}M_1$$

FIG. 69

supported. The first three spans and the last span are loaded with a uniformly distributed load, w lb. per ft. The fourth span is unloaded.

The first line represents the moments at the respective joints, with all joints except Joint F locked. The fixed end moment, $\dfrac{wl^2}{12}$, at Joints A, B, C, and D, is represented by 100, while $\dfrac{wl^2}{8}$ at Joint E is represented by 150. The second line represents the effect of unlocking Joints B and C. Since



$$k_1 = \frac{E_1I_1}{l_1};\quad k_2 = \frac{E_2I_2}{l_2};\quad k_3 = \frac{E_3I_3}{l_3}$$

$$M_1 = \frac{3k_1(4k_1k_2 + 4k_2k_3 + 4k_2^2 + 3k_1k_3)}{3k_2^2 + 4k_1k_2 + 3k_2k_3 + 3k_1k_3}$$

$$M_2 = \frac{2k_2(k_2 + k_3)M_1}{4k_1k_2 + 4k_2k_3 + 4k_2^2 + 3k_1k_3}$$

$$M_3 = \frac{k_3M_2}{2(k_2 + k_3)}$$

When $k_1 = k_2 = k_3$ then $M_1 = \dfrac{45EI}{13l}$;

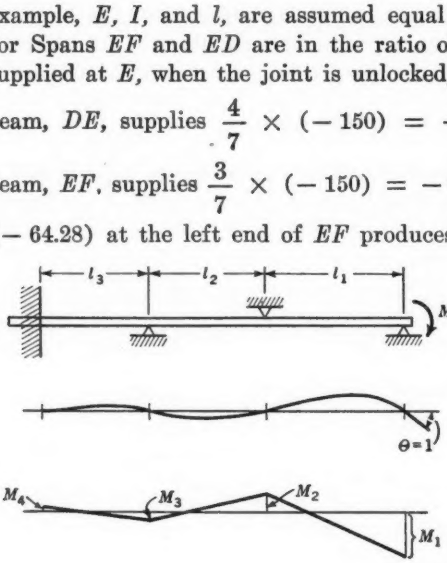
$$M_2 = \frac{4}{15}M_1;$$

$$M_3 = \frac{M_1}{15}$$

FIG. 70

there are no unbalanced moments at these points, no elastic deformations will take place when these joints are unlocked. Next, consider unlocking Joint E, keeping in mind that Joint D is still considered as locked. Fig. 66 provides the key for the stiffness factor of Span EF, while Fig. 67 provides the key for the stiffness factor of Span ED. Let M_1 in each case represent the moment necessary to produce unit angular displacement. Since, in this

example, E, I, and l, are assumed equal for all spans, the stiffness factors for Spans EF and ED are in the ratio of 3 to 4. The total moment to be supplied at E, when the joint is unlocked, is (-150). The right end of the beam, DE, supplies $\frac{4}{7} \times (-150) = -85.72$, while the left end of the beam, EF, supplies $\frac{3}{7} \times (-150) = -64.28$. This increment of moment (-64.28) at the left end of EF produces no moment effect at the point, F,



$$M_1 = \frac{4k_1(3k_2^2 + 4k_2k_3 + 3k_1k_2 + 3k_1k_3)}{3k_2^2 + 4k_2k_3 + 4k_1k_2 + 4k_1k_3}$$

$$M_2 = \frac{k_2(3k_2 + 4k_3)\,M_1}{2(3k_2^2 + 4k_2k_3 + 3k_1k_2 + 3k_1k_3)}$$

$$M_3 = \frac{2k_3}{3k_2 + 4k_3}M_2$$

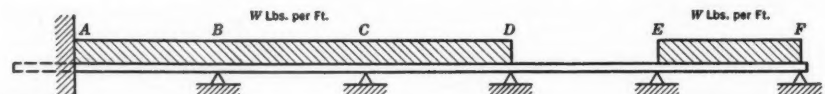
$$M_4 = \frac{M_3}{2}$$

When $k_1 = k_2 = k_3$ then $M_1 = \frac{52\,EI}{15\,l}$; $M_2 = \frac{7}{26}M_1$; $M_3 = \frac{2}{26}M_1$; $M_4 = \frac{1}{26}M_1$

FIG. 71

which was from the start a pin-connected joint. The moment, -85.72, at the right end of DE, induces a moment of $\frac{1}{2} \times (-85.72) = -42.86$ at D. These values, induced by the unlocking of Joint E, are recorded on Line 3, Fig. 72.

The only joint that remains to be unlocked is the joint, D. The stiffness factors for the right and left parts of the structure are taken from Fig. 68 and Fig. 71. In this case, in which the values of $k = \frac{EI}{l}$ for all spans are equal, the respective stiffness factors for DF and DA are



	A	B	B	C	C	D	D	E	E	F
All Joints except F, are Locked	+100	−100	+100	−100	+100	−100	0	0	+150	0
Unlock B and C		0	0	0	0					
Unlock E							−42.86	−85.72	−64.28	0
Unlock D	+2.76	+5.52	−5.52	−19.34	+19.34	+71.84	+71.02	+17.75	−17.75	0
Resultant Moments at All Joints	102.76	94.48		119.34		18.16		67.97		0

FIG. 72

$\frac{24}{7}$ and $\frac{52}{15}$, or 3.428 and 3.467. As Joint D is unlocked, a total balancing moment of $+142.86$ is to be supplied. The left end of DF supplies $\frac{3.428}{3.428 + 3.467} \times (+142.86) = +71.025$, while the right end of AD supplies 71.84. These values are recorded under D on the fourth line of Fig. 72.

The effect of change in the moment at Joint D on the other joints can be obtained directly from Figs. 68 and 71. Thus, $\frac{7}{26} \times 71.84 = 19.34$ is the effect on C of the 71.84-moment change at D. The effect on Joint E of the moment change of 71.02 at Joint D is obtained from Fig. 68. This amounts to $\frac{1}{4} \times 71.02 = 17.75$. The final moments are obtained by adding the values under each joint. The answers in this case involve no approximation other than that involved in the number of significant figures to which the computations are carried. Theoretically, one may provide oneself with any number of key diagrams. It is thought that the six key diagrams offered herewith (Figs. 66 to 71) are sufficient for most practical problems. Fig. 71 shows that for a beam of three equal spans, $M_4 = \frac{1}{26} M_1$. This suggests that the moment change need not be distributed much beyond three spans, if at all. Approximation occurs only when one ignores the effect of a

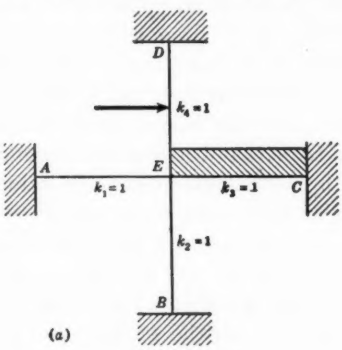


FIG. 73

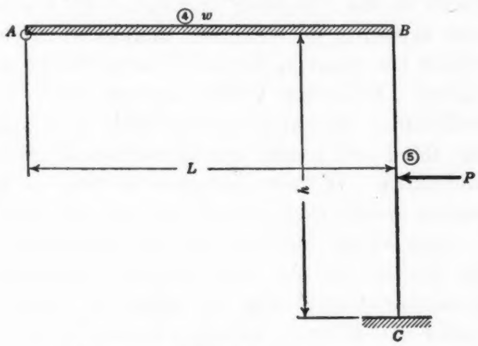


FIG. 74.

moment change at some joint on another joint so far removed that a key diagram is not available, or when the one available is not used because it is known that the effect involved is negligible.

Consider Fig. 73: Assume Joint E as locked, and let M_{EC} be represented by $+ 100$, and M_{CE} by $- 100$; let $M_{ED} = + 60$; $M_{DE} = - 60$; and $M_{EA} = M_{AE} = M_{EB} = M_{BE} = 0$. If $k_1 = k_2 = k_3 = k_4$, then, as the unbalanced moment at E is $+ 160$, each member supplies an equal balancing moment, which is $\frac{- 160}{4} = - 40$. The key diagram (Fig. 67) applies to all four legs. Thus, one-half the balancing moments is carried along each member to its extremity. The answers, then, are: $M_A = - 20$; $M_B = - 20$; $M_C = - 100 - 20 = - 120$; $M_{DE} = - 60 - 20 = - 80$; $M_{EC} = +100 - 40 = + 60$; $M_{ED} = + 60 - 40 = + 20$; $M_{EA} = - 40$; and $M_{EB} = - 40$.

Suppose k_1, k_2, k_3, and k_4 are in the ratio, 1:2:3:4; they, as Joint E, Fig. 73, is unlocked, $M_{EA} = \frac{1}{10} \times (- 160) = - 16$; $M_{AE} = \frac{1}{2} \times (- 16) = - 8$; $M_{EB} = \frac{2}{10} \times (- 160) = - 32$; and $M_{EC} = + 100 + \frac{3}{10} \times (- 160) = + 52$.

Suppose Joint C in Fig. 73, is pin-connected, and $k_1 = k_2 = k_3 = k_4$. With the same loading as shown, when Joint E is assumed to be locked, while Joint C is pin-connected, $M_{EC} = +150$. The unbalanced moment at Joint E then is $+ 150 + 60 = + 210$. The stiffness factor for EC now is $\frac{3EI}{l}$, while, for the other three legs, it is $\frac{4EI}{l}$. The ratio, therefore, is 3 to 4. As Joint E is now unlocked, $M_{EA} = \frac{4}{15} \times - 210 = - 56$; $M_{AE} = - 28$; $M_{EC} = + 150 + \frac{3}{15}(-210) = + 108$; $M_{CE} = 0$; $M_{ED} = + 60 + \frac{4}{15}(-210) = + 4$; and $M_{DE} = - 60 + \frac{1}{2} \times \frac{4}{15} \times (-210) = - 88$.

The writer's engineering vocabulary does not include the much-loved word "exact"; otherwise, he would feel like applying this adjective to the answers given in the foregoing example. He has little objection to approximations, and certainly no objection thereto worth mentioning in design problems in which the number, 2, is as low as designers dare go in selecting a factor of safety. Professor Cross suggests that two approximations are generally sufficient. However, in his Table 1, it appears that the increment between the third and fourth approximation is greater than that between the second and third. If there is reason to carry on to three approximations, the same reason would then compel one to continue to at least four approximations.

The writer does not want to take space to work Professor Cross' example by the method he has herewith suggested. Joints B, D, E, and F, of Professor Cross' Fig. 1, might be unlocked successively. The stiffness factor of CBA may be taken from Fig. 69. Fig. 66 gives the stiffness factor for CF; Fig. 67 gives the stiffness factor for CG; while Fig. 68 gives the stiffness factor for CDE. After Point C is balanced, the carried-over moments for B and A may be taken from Fig. 69, while that for D is taken from Fig. 68. The factor carried over to F is zero, while the factor carried over to G is taken from Fig. 67. This procedure completes the entire solution. The results thus obtained check the values given by Professor Cross.

One more example might be given. In the **L**-frame, Fig. 74, the horizontal leg, AB, pinned at A, is 25 ft. long, and is loaded with 1 000 lb. per ft. (see, also, Fig. 5). Considering Joint B as fixed, its fixed-end moment, M_{BA}, would be: $\frac{wl^2}{8} = \frac{1\,000 \times 25^2}{8} = - 78\,125$ ft-lb.. The vertical leg, BC, fixed at C, and 20 ft. high, is loaded with a concentrated load of 20 000 lb. at its center. The fixed-end moment, $M_{BC} = \frac{Ph}{8} = \frac{20\,000 \times 20}{8} = 50\,000$ ft-lb. The unbalanced moment at C is $- 28\,125$ ft-lb. The elastic coefficient, $\frac{I}{l}$, for BA is 4, while for BC it is 5. The stiffness factor for BA from Fig. 66 is $\frac{3\,EI}{l}$

$= 3 \times 4\,E = 12\,E$, while that for BC from Fig. 67 is $\dfrac{4\,EI}{l} = 4 \times 5 \times E$ $= 20\,E$. As the joint is unlocked, the unbalanced moment distributes itself between BA and BC in the ratio of 12 to 20.

The final moment, $M_{BA} = -\,78\,125 + \dfrac{12}{32} \times 28\,125 = -\,67\,578$ ft-lb. The final moment, $M_{BC} = +\,50\,000 + \dfrac{20}{32} \times 28\,125 = +\,67\,578$ ft-lb. which compares with the value of 67 360 ft-lb. as determined by the more formal procedure.[48b] Once familiar with this method, one obtains in round figures the answer to a problem as simple as the foregoing without putting a figure on paper.

[48b] *Proceedings*, Am. Soc. C. E., September, 1930, Papers and Discussions, p. 1757.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

HIGHWAY LOCATION: PRACTICAL CONSIDERATIONS

Discussion

BY A. R. LOSH, ASSOC. M. AM. SOC. C. E.

A. R. LOSH,[7] Assoc. M. Am. Soc. C. E. (by letter).[7a]—The writer appreciates the favorable comments his paper has received from members of the Society who have had extensive experience in National and State highway programs. The information and suggestions contributed have added materially to the discussion.

The importance of conditions which will reduce or eliminate the difficulties of snow drifting was stressed by Mr. Bishop. This is worthy of careful attention not only from the viewpoint of reducing the cost of snow removal, but, as Mr. Bishop states, to keep the roads open to the public. The cost of snow removal during the winter of 1929-30 as given by Mr. Bishop was $8 250 000 for 185 000 miles of road, or an average of $44.50 per mile. Considering the severe conditions of that season this is not high. The cost of snow removal on Oklahoma highways for the same season was an average of $30.00 per mile. The loss to the State in gasoline taxes alone was more than three times the cost of snow removal, due to the fact that extreme winds created drifts along the roads three times after they had been broken open. The real loss to the traveling public is, of course, unknown. In the plains country fine dry snow carried by a winter gale will drift into every cut and depression. Until the snow has settled and partly packed, it is difficult to keep an east and west road open. Freedom from snow hazard is an important location consideration in places subject to heavy snowfall.

Referring to tourist roads, parkways, scenic highways, and places of historic or recreational interest, Mr. Bishop points out the necessity of departing from the engineering phases of grades and alignment in location studies and giving due weight to those features which determine the general nature

NOTE.—This paper by A. R. Losh, Assoc. M. Am. Soc. C. E., was presented at the Joint Meeting of the Highway and Construction Divisions, Dallas, Tex., April 25, 1929. and published in November, 1930, *Proceedings*. Discussion on the paper has appeared in *Proceedings* as follows: February, 1931, by H. K. Bishop, M. Am. Soc. C. E.; and May, 1931, by Messrs. J. C. Carpenter, H J. Spelman, H. S. Kerr, and H. W. Giffin.

[7] Mgr., Asphalt and Road Oil Div., Anderson-Prichard Corp., Oklahoma City, Okla.

[7a] Received by the Secretary January 4, 1932.

of the road. Conditions such as these are unusual in highway location, but their importance has been properly stressed by Mr. Bishop.

The advancement of highway location standards resulting from the Federal Aid acts is well brought out by Mr. Carpenter. He also makes the point that public sentiment too often must be given consideration on detail location. Of particular value is his suggestion that a major location plan be worked out for future improvement, and that this be utilized on individual sections as the opportunity arises. His solution of the "main street" location for the small city is a practical one frequently used to advantage.

The improvement of the motor vehicle and the higher permissible speeds of recent years have brought about the necessity of better location, as stated by Mr. Spelman. Unless, as he suggests, highway engineers locate more boldly and with a freer hand than in the past, 1931 locations will be obsolete a few years hence.

Mr. Kerr's discussion referring to the work of the Utah State Highway Department brings out a number of general conditions to be met in highway location. First is the necessity of solving the political and economic problems as was done in the earlier locations of that State when the towns and cities were provided with service along somewhat local lines. Having met this local pressing need the Department could work with a freer hand in developing the transcontinental routes. The trend of public opinion was for the cross State routes once the immediate needs of the populous sections had been met. This later work permitted the exercise of engineering skill to the fullest extent. Fortunately, the more expensive construction came at a time when the location was possible on engineering features.

Public support is a necessity in any highway program. This statement by Mr. Giffin is borne out in Mr. Carpenter's discussion, and also by Mr. Kerr. While it is highly desirable to locate for the future, present problems must be met. The public can be led to a certain extent, but the leader must not get too far ahead of the procession. The writer agrees fully with Mr. Giffin in that every condition on the highway which requires special action on the part of the operator should be brought to his attention in ample time for him to control the vehicle; and, as he further states, speed is the important factor to consider. A curve of 50-ft. radius was no hazard for vehicles moving at 8 miles per hour; but a 500-ft. radius is a serious condition for speeds greater than 35 miles per hour. In answer to Mr. Giffin's query, the writer feels that anticipated speeds of 70 miles per hour are not improbable ten years hence. Traffic is moving at nearly this speed in the open country now (1932), and at too great a hazard from intersecting highways at grade, narrow structures, embankments of insufficient width, and other conditions which the engineer must eliminate in the future.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

RUN-OFF—RATIONAL RUN-OFF FORMULAS

Discussion

BY MESSRS. RALPH W. POWELL, MERRILL M. BERNARD, AND CHARLES H. LEE

RALPH W. POWELL,[45] M. AM. SOC. C. E. (by letter).[45a]—This paper has been quite fully discussed and the writer wishes to touch on only a few points. The authors derive a formula for C, the run-off coefficient. It would seem that if z is the rate of absorption plus evaporation, in cubic feet per second per acre, the total number of cubic feet per second being absorbed and evaporated would be zA, the net run-off would be $iA - miA - zA$, and C would be $1 - m - \frac{z}{i}$, instead of as in Equation (39). The authors seem to have allowed for no absorption or evaporation from the stored portion of the precipitation.

The storage here referred to is that "falling in depressions, cavities, etc.," and evidently is not meant to include "channel storage." The writer questions whether Kutter's formula, or any other formula derived from experiments on steady flow, applies to this case. It would seem that the channel must act to some extent like a retarding basin and make the time of concentration for the uniform rainfall assumed by the authors, somewhat more than that given by their method. If this is true the error, of course, would be on the side of safety.

Actual storms are not like the one assumed, however, and they do not occur simultaneously over the water-shed. There is the extreme possibility of a storm moving down a valley at the same velocity as the flood crest. This would mean that the maximum effects of the most intense rainfall in each part of the drainage area would reach the mouth simultaneously and that the maximum flow would be $Q = Cia$, in which, i would be the intensity for

NOTE.—The paper by R. L. Gregory and C. E. Arnold, Associate Members, Am. Soc. C. E., was published in April, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. Le Roy K. Sherman, Francis Bates, and John W. Raymond, Jr.; November, 1931, by Messrs. Reginald A. Ryves, G. S. Tapley, W. I. Hicks, John M. Kemmerer, Carl H. Reeves, Leonard L. Longacre, G. H. Hickox and Donald M. Baker; December, 1931, by Albert R. Arledge, Assoc. M. Am. Soc. C. E.; January, 1932, by Clarence S. Jarvis, M. Am. Soc. C. E.; and February, 1932, by C. E. Grunsky, Past-President, Am. Soc. C. E.

[45] Asst. Prof. of Mechanics, Ohio State Univ., Columbus, Ohio.

[45a] Received by the Secretary December 30, 1931.

a very short time, t, approaching zero as the timing of the maximum rainfall at each point in the area approached the ideal just stated. This would mean values of i of the order of 10 in. per hour. It would seem, therefore, that peak run-off resulting from any given storm depends too vitally upon the velocity (including direction) of the storm across the drainage area to make worth while any refinements of methods which neglect it.

Speaking of small values of t it must be remembered that Equation (2) and all the information compiled[46] by Merrill M. Bernard, M. Am. Soc. C. E., apply only to times greater than 2 hours. For shorter times, the formula must be changed to,

$$i = \frac{k}{(t + b)} e \qquad (113)$$

in which, b is about 7 or 8 min.[47]

On the other hand, when the area becomes large another correction is necessary as pointed out by the authors. From the study of a small part of the data given in the Appendix of Part V of the Technical Reports of the Miami Conservancy District, the writer has derived a rough empirical correction factor of the form, $(1 - 0.015 \sqrt[3]{A_m})$, in which, A_m is the area, in square miles. This gives values nearly the same as those in Table 15,[47a] but was derived independently. It is evident that this correction should be made whenever A_m is more than, say, 100 sq. miles.

Merrill M. Bernard,[48] M. Am. Soc. C. E. (by letter).[48a]—The authors' paper may be accepted as a definite step toward the goal of determinate run-off values. With it they have provided new tools with which to attack the problem of rainfall-run-off relationships.

The limitations of the rational theory must first be accepted and its field of application defined before it can become the basis of intelligent design. A principal drawback is, that the magnitude and frequency of a storm—the run-off from which brings with it the run-off of previous storms, detained in storage as snow and ice—cannot in any way become the measure of the magnitude and frequency of the resulting flood flow. The writer, therefore, would limit the application of his suggestions in the following discussion to that part of the Eastern United States south of the snow line, and to summer conditions only, for the northern section.

Too little emphasis has been laid on the frequency with which flood magnitude and duration may be expected. This factor becomes the measure of efficiency of any structure, the purpose of which is to remove or pass flood waters. Drainage structures may be divided into two general classes: First, conduits or channels, for the removal of storm run-off; and, second, spillways and important bridge openings.

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[46] *Proceedings*, Am. Soc. C. E., October, 1931, pp. 1276–1280.

[47] "Frequency and Intensity of Excessive Rainfalls at Boston, Massachusetts," by Charles W. Sherman, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 954, and the discussion by M. F. Wagnitz, M. Am. Soc. C. E., and L. C. Wilcoxen, Assoc. M. Am. Soc. C. E., p. 964.

[47a] *Proceedings*, Am. Soc. C. E., November, 1931, p. 1416.

[48] Cons. Civ. Engr., Crowley, La.

[48a] Received by the Secretary February 11, 1932.

In economical design the first class demands a balance between cost and capitalized damage resulting from prolonged inundation, the damage being understood to include that to private property, hazard to health, and curtailed convenience to the public.

In addition to property damage the structures of the second class involve hazard to life. In that case design must provide capacity to meet a maximum condition or, in terms of frequency, flood intensities reached or exceeded once in from 100 to 200 years. Design, based on an isolated storm intensity which is classed as "maximum" in a short-term rainfall record, could be dispensed with in favor of direct application of personal judgment to the size of the structure itself.

It is obvious that the frequency of the storm is not necessarily the frequency of the flood produced by it. A highly intense storm of long duration, to be expected not oftener than once in 25 years, may occur at a time following a period of little or no rainfall. Then, especially on open, pervious water-sheds, the demands of the losses to run-off are high, resulting in a low value of the coefficient, C, and a unit run-off that, were flow records available over a sufficiently long period, might be found to have a frequency of once in 5 years. Flood frequency will approach storm frequency, however, because the outstanding storms of record have produced flood flows which must be nearly as infrequent in occurrence.

Failure to advance in this natural science cannot be laid altogether to blind faith in the old and accepted empirical formulas, but in the fact, as the authors have stated, that there has been no uniformity in the conception of assumptions made in the present use of the rational method. Confusion in an understanding of its application to rural areas and stream-flow analysis has arisen where the terms in the equation, $Q = C\,i\,A$, have been given meanings applicable only in the field of storm sewer design, or where the terms have been inadequately or obscurely defined.

Particularly does there seem to be no uniformity in the understanding of the run-off coefficient, C. It is usually defined as the ratio of the rate of run-off to that of rainfall, which may take any of the following meanings:

(*a*) Total run-off to total rainfall for the entire storm period.
(*b*) Maximum rate of run-off to maximum rate of rainfall.
(*c*) Rate of run-off at time of concentration to average rate of rainfall at time of concentration.
(*d*) Maximum rate of run-off to average rate of rainfall throughout a duration period equal to time of concentration.

The rational theory assumes that the rate of travel of the flood crest is the same as that of a particle of water leaving the remote (in terms of time), portion of the water-shed. This assumption may be met in the city storm sewer system, but on the large, agricultural water-shed, it must be modified to recognize a lag between time of concentration, as established by the authors' run-off equation, or by estimated velocities and the arrival of the flood crest. The hydrologist has no interest in the rate of run-off prevailing at the end of the calculated concentration period, but in a maximum rate that

will occur later. The writer believes, also, that to burden an already laborious procedure with the complications of non-uniform flow, involved in the travel of the flood crest, is out of the question. Since designers are interested only in maximum discharge and, since it is practicable to determine a concentration period, from determinate factors, representing time in transit from headwaters to concentration point, the coefficient, C, should be understood to be the ratio of maximum discharge to the average rate of rainfall intensity throughout a duration period equal to such time of concentration.

Concentration time should be understood to be a time interval, at the end of which all effective parts of the water-shed are contributing to the flow at the point of concentration. It should be conceived as a time contour, developing around the point of concentration, and progressively receding to the limits of the water-shed. This will exclude artificial reservoirs and appreciable areas of swamp, lake, or marsh, in which the run-off waters because of restricted outlet or other cause, will not reach the point under consideration until the condition of maximum flow has been reached and has passed.

While the theory is stated as a simple basic relationship, $Q = C\,i\,A$, the method of application and the procedure change continuously as the area embraced increases. Conceive of a water-shed, the outfall channel of which heads as a city storm sewer system, discharging into a local stream which, after receiving the drainage waters from an agricultural area, empties into a river which, in turn, becomes the tributary to a still larger stream. Three distinct problems are met involving cases in which run-off is produced: First, by short intense local showers; second, by a single widespread storm or a combination of such storms occurring nearly at the same time; and, third, by the melting of the winter's snow or protracted wet spells, or by their coincident occurrence, over large river basins. In each case the factors affecting the ratio of run-off to rainfall rate, except for a common name and symbol, are different in character and in the degree to which they influence that ratio.

An intense downpour of 200 min. or less, critical to the storm sewer system, becomes only one of several phases of a storm period producing run-off from the larger areas. The average rainfall intensity prevailing throughout the storm phase may not be far less than its maximum in value, while the average of an extended storm period will not approach the storm maximum; and as duration continues it will vary between much narrower limits. It is significant that the expression for rainfall intensity changes rather abruptly at the end of one or two hours. The equation,

$$i = \frac{a}{t + b} \qquad \text{(114)}$$

may be taken as representing the comparatively short period of continuous downpour; and the equation,

$$i = \frac{K}{t^e} \qquad \text{(115)}$$

for the longer durations, representing the average of intermittent rainfall throughout the storm period.

Another difference lies in the fact that individual storms may arrange themselves in many ways, peaking in the early stages, in the middle, or at the end of the storm period. An intensity curve group will give the relation between rainfall intensity, duration, and frequency for a locality, but it cannot account for the variation in average intensity at intervals throughout the individual storm.

Fig. 12 shows nine storm curves[49] plotted over the rainfall intensity curve group[50] for Central Illinois. While there is no apparent order in their early stages, they straighten out, after about 400 min., and in general follow the law of the curve group. The shape of the storm curve, materially affecting

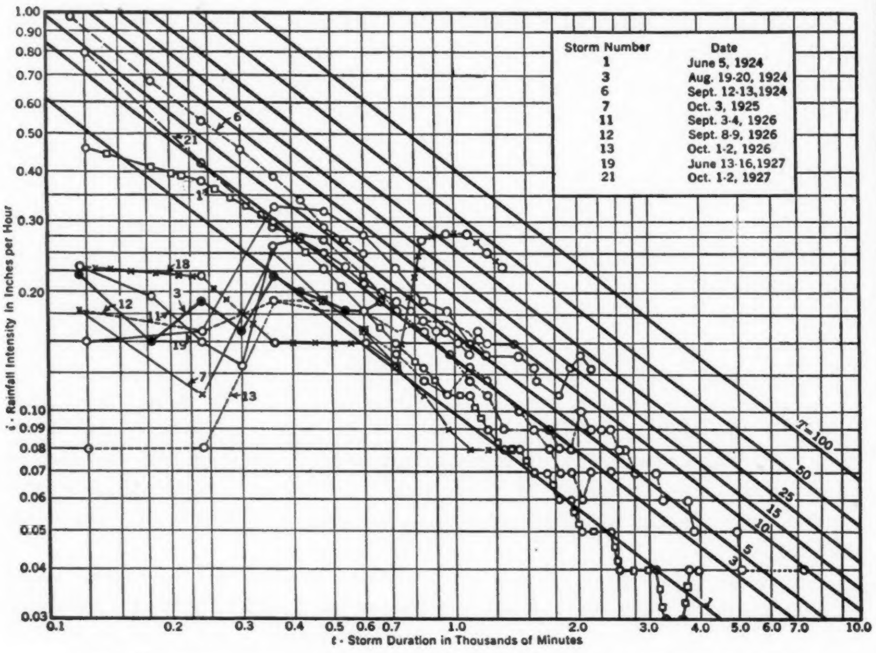

FIG. 12.—COMPARISON OF STORM CURVES WITH RAINFALL INTENSITY CURVE, GROUP FOR CENTRAL ILLINOIS $\left(i = \frac{23\,T^{0.2}}{t^{0.79}}\right)$.

velocities, then concentration time, and, finally, the selection of the controlling rainfall intensity, is of relatively high importance to the storm sewer system, when concentration periods are short, but it reduces in importance for large areas having concentration time of many hours.

There seems to the writer no justification in extending the use of the rational method, in its present state of development, to flood-flow problems covering several thousand square miles. The treatment of such cases requires a rainfall intensity distribution factor, as yet not fully developed, and the

[49] "Run-Off Investigations in Central Illinois," U. S. Bureau of Public Roads, and Bulletin *232*, Univ. of Illinois Eng. Experiment Station, 1931.

[50] "Formulas for Rainfall Intensities of Long Duration," by Merrill M. Bernard, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., October, 1931, p. 1275.

factors affected by topography and soil conditions must become carefully weighted averages. Furthermore, it is probable that the coefficient, C, will take an entirely new range of values.

The authors' paper, with the writer's rainfall intensity study covering the Eastern United States,[51] and data, recently gathered in detail for Central Illinois,[49] present an opportunity to develop a complete example of the application of the rational theory to agricultural drainage and stream-flow problems.

For the Eastern United States and for duration periods greater than 1 hour, the general formula for Q (Equation (24)), may be expressed as:

$$Q = U\ (C\ A\ K)^{4g}\ T^{4xg} \quad \text{..................(116)}$$

in which K, T, and x are the intensity coefficient, frequency coefficient, and frequency exponent, respectively, taken from the writer's charts, all other notations being the same as in the authors' paper. The symbol, U, becomes a factor reflecting water-shed characteristics, in which, e is the exponent of duration, t, thus,

$$U = \left(\frac{60\ P}{L}\right)^{4eg} F^{8eg}\ \frac{S^{1.5eg}}{1\ 000^{2eg}} \quad \text{................(117)}$$

The formula may be reduced by using the average value of e for the Eastern United States, which varies from 0.75 to 0.83, as 0.80. Then,

$$Q = U\ (C\ A\ K)^{1.25}\ T^{1.25x} \quad \text{..................(118)}$$

and,

$$U = 1.897\ (B\ F)^2\ S^{0.375} \quad \text{..................(119)}$$

In this form, Equation (119) lends itself readily to the inclusion of other factors (such as one for rainfall intensity distribution) and permits the rapid comparison of Q for various frequencies, which is a decided advantage to design. It will be found that the Illinois data have provided values for all factors in the formula, as shown in Table 16.

The rainfall intensity formula for this locality is,

$$i = \frac{23\ T^{0.30}}{t^{0.79}} \quad \text{........................(120)}$$

and the run-off equation is,

$$Q = U\ (C\ A\ K)^{1.246}\ T^{0.374} \quad \text{..................(121)}$$

in which,

$$U = \left(\frac{60\ P}{L}\right)^{0.984} F^{1.969}\ \frac{S^{0.369}}{29.9} \quad \text{................(122)}$$

Transposing,

$$C_a = \left(\frac{Q}{U\ (A\,K)^{1.246}\ T^{0.374}}\right)^{0.803} \quad \text{................(123)}$$

[51] "Formulas for Rainfall Intensities of Long Duration," by Merrill M. Bernard, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., October, 1930, p. 1835, and subsequent discussions.

It will be found that, as long as T and i are in accord with the law of the curve group, the time of concentration will remain constant. The values in Columns (5), (6), (7), (8), (9), (12), (13), (14), (15), and (16), in Table 17, were obtained by the following procedure:

TABLE 16.—DATA COMPILED FROM RUN-OFF INVESTIGATIONS IN CENTRAL ILLINOIS

No. (1)	Water-Shed (2)	Area, in acres (3)	Maximum channel length, in feet (4)	Channel slope, in feet per 1 000 ft. (5)	W (6)	$\frac{L}{W}$ (7)	P_1 (for uniform rainfall intensity) (8)	$\frac{60\ P_1^{0.984}}{L}$ (9)
1	Fountain Head	2 982	22 700	4.65	5 720	3.975	0.49	0.00141
2	St. Joseph	3 315	17 400	2.27	8 380	1.743	0.44	0.00150
3	Lateral No. 15	7 360	41 200	0.85	7 790	5.285	0.50	0.000819
4	Kaskaskia Mutual	7 680	28 500	4.36	11 720	2.435	0.45	0.00101
5	Camp Creek	9 792	42 200	3.03	10 100	4.185	0.49	0.000784
6	Two-Mile	9 792	35 900	2.08	11 870	3.025	0.47	0.000833
7	Embarrass River, east of Tolon	17 408	50 700	2.08	14 960	3.427	0.48	0.000642
8	Kaskaskia River	36 864	90 800	1.70	17 690	5.065	0.50	0.000376
9	West Branch, Salt Creek	38 720	83 400	0.87	20 220	4.120	0.49	0.000402
10	Lake Fork	40 448	100 800	0.66	17 550	5.721	0.51	0.000346

No.	Water-Shed	P_2 (for rainfall intensity varying with $t^{0.5}$) (10)	$\frac{60\ P_2^{0.984}}{L}$ (11)	Side slopes (12)	Bottom to depth ratio (13)	Kutter's n (14)	F (15)	$F^{1.964}$ (16)
1	Fountain Head	0.55	0.00162	1 on 3	1 : 1	0.03	4.83	22.16
2	St. Joseph	0.48	0.00183	1 on 3	1 : 1	0.10	3.08	9.14
3	Lateral No. 15	0.57	0.000933	1 on 1	1 : 1	0.05	4.65	20.56
4	Kaskaskia Mutual	0.51	0.00120	1 on 1	1 : 1	0.10	3.32	10.60
5	Camp Creek	0.55	0.000881	1 on 2	2 : 1	0.05	4.45	18.86
6	Two-Mile	0.53	0.000994	1 on 2	2 : 1	0.10	3.18	9.74
7	Embarrass River, east of Tolon	0.54	0.000722	1 on 2	2 : 1	0.03	4.97	23.44
8	Kaskaskia River	0.57	0.000425	1 on 2	2 : 1	0.04	4.65	20.56
9	West Branch, Salt Creek	0.55	0.000451	1 on 2	2 : 1	0.04	4.65	20.56
10	Lake Fork	0.58	0.000395	1 on 2	2 : 1	0.09	3.40	11.11

First.—A value of the drainage coefficient for $T = 1$ year was calculated by Equation (123);

Second.—The 1-year i was obtained by dividing the unit, q, taken from the storm record, by the 1-year C;

Third.—Concentration time was established by taking the 1-year i into the curve group to an intersection with the 1-year frequency curve;

Fourth.—The average rainfall intensity for the duration equal to concentration time, as determined by the first, second, and third steps, was taken from the individual storm curve;

Fifth.—This intensity, taken into the curve group with its time of duration, established its frequency; and,

Sixth.—The unit, q, was divided by the prevailing i to arrive at the actual value of the drainage coefficient, shown as C_a in Columns (9) and (16), Table 17, for each storm and water-shed.

TABLE 17.—COMPARATIVE EFFECT OF P_1 AND P_2 ON OTHER FACTORS THAT AFFECT RUN-OFF

Water-Shed No. (1)	Storm No. (2)	Date (3)	max.q (4)	t, min. (5)	i (6)	T_l (7)	U (8)	Ca (9)	z (10)	Cb (11)	t, min. (12)	(13)	T (14)	U (15)	Ca (16)	z (17)	Cb (18)
1.......	2	Oct., 1926	0.0450	179	0.300	0.5	0.001830	0.150	0.140	0.151	157	0.344	0.5	0.002105	0.133	0.186	0.133
1.......	3	June, 1924	0.1340	137	0.485	1.0	0.001830	0.276	0.012	0.277	118	0.480	0.8	0.002105	0.279	0.020	0.278
2.......	4	Oct., 1925	0.0118	730	0.128	1.1	0.000624	0.092	0.086	0.093	600	0.160	1.4	0.000756	0.074	0.120	0.073
2.......	5	Sept., 1926	0.0202	655	0.181	2.5	0.000624	0.112	0.109	0.113	525	0.203	2.1	0.000756	0.100	0.130	0.104
2.......	6	Oct., 1926	0.0299	580	0.280	8.0	0.000624	0.106	0.175	0.107	480	0.330	8.3	0.000756	0.090	0.225	0.092
3.......	7	Sept., 1926	0.0512	485	0.180	1.2	0.000529	0.284	0.000	0.284	425	0.185	0.9	0.000604	0.277	0.008	0.278
3.......	8	Oct., 1927	0.0153	670	0.082	0.2	0.000529	0.187	0.028	0.187	580	0.093	0.2	0.000604	0.165	0.040	0.165
3.......	9	Oct., 1926	0.0562	480	0.240	2.8	0.000529	0.234	0.041	0.235	420	0.240	2.0	0.000604	0.234	0.045	0.236
4.......	10	June, 1924	0.0231	510	0.238	3.1	0.000616	0.097	0.155	0.099	425	0.295	4.2	0.000733	0.079	0.212	0.081
4.......	11	Oct., 1926	0.0222	510	0.195	1.6	0.000616	0.114	0.116	0.115	430	0.202	1.3	0.000733	0.110	0.122	0.113
4.......	12	Aug., 1924	0.0285	480	0.150	0.6	0.000616	0.190	0.049	0.191	400	0.150	0.4	0.000733	0.190	0.050	0.193
5.......	13	June, 1924	0.0168	420	0.325	5.6	0.000747	0.052	0.265	0.051	375	0.360	5.6	0.000839	0.047	0.302	0.047
5.......	14	Oct., 1927	0.0228	390	0.285	2.9	0.000747	0.080	0.204	0.081	350	0.300	2.6	0.000839	0.076	0.220	0.077
5.......	15	Aug., 1924	0.0264	380	0.153	0.4	0.000747	0.173	0.059	0.174	340	0.162	0.3	0.000839	0.163	0.071	0.163
6.......	16	Sept., 1926	0.0154	940	0.155	3.7	0.000350	0.100	0.100	0.101	780	0.122	1.1	0.000423	0.126	0.068	0.128
6.......	17	July, 1927	0.0146	945	0.175	5.9	0.000350	0.083	0.122	0.086	790	0.240	10.5	0.000423	0.061	0.188	0.063
6.......	18	Oct., 1926	0.0311	790	0.130	1.4	0.000350	0.239	0.020	0.240	650	0.143	1.2	0.000423	0.217	0.038	0.213
7.......	19	June, 1924	0.0230	390	0.300	3.5	0.000658	0.077	0.219	0.077	355	0.321	3.5	0.000740	0.072	0.288	0.073
7.......	20	July, 1927	0.0437	330	0.280	1.8	0.000658	0.156	0.126	0.156	295	0.230	0.7	0.000740	0.190	0.076	0.190
7.......	21	Oct., 1926	0.0671	305	0.359	3.2	0.000658	0.187	0.120	0.189	262	0.410	3.3	0.000740	0.164	0.171	0.165
8	22	June, 1927	0.0150	750	0.125	1.0	0.000315	0.120	0.071	0.123	665	0.132	0.9	0.000357	0.114	0.080	0.114
8.......	23	July, 1927	0.0197	700	0.338	25.0	0.000315	0.058	0.269	0.058	620	0.326	15.0	0.000357	0.060	0.258	0.161
8.......	24	Oct., 1926	0.0296	640	0.140	1.0	0.000315	0.211	0.036	0.211	560	0.140	0.7	0.000357	0.211	0.039	0.209
9.......	25	June, 1924	0.0238	800	0.143	1.9	0.000262	0.166	0.059	0.167	705	0.153	1.8	0.000294	0.156	0.070	0.157
9.......	26	Sept., 1926	0.0128	950	0.162	4.6	0.000262	0.079	0.116	0.081	840	0.180	4.8	0.000294	0.071	0.135	0.072
9.......	27	Oct., 1926	0.0252	790	0.128	1.4	0.000262	0.197	0.039	0.197	700	0.143	1.4	0.000294	0.176	0.053	0.182
10.......	28	June, 1924	0.0078	2 530	0.040	0.6	0.000109	0.195	0.012	0.199	2200	0.048	0.7	0.000125	0.162	0.021	0.163
10.......	29	Oct., 1926	0.0178	2 070	0.084	4.0	0.000109	0.213	0.021	0.213	1800	0.032	4.3	0.000125	0.188	0.030	0.180

Comparative results, using two values of the factor, P, are given in Table 17, the former, with P_1, assuming rainfall intensities as uniformly distributed, and the latter, with P_2, assuming rainfall intensities as following the curve of the type, $i = \frac{K}{t^{0.5}}$.

It would appear to the writer that opportunity for a condition to occur in accordance with the use of P_2, that is, a storm that comes to a peak in the first 20 to 60 min. of its duration and progressing down stream (thus shortening concentration time by building up high unit discharges and velocities at the head-waters), will be much more infrequent than an average condition, represented by P_1, the assumption of a uniform intensity over the water-shed above the point of concentration. Table 17 compares the effect of P_1 and P_2 on the other factors affecting run-off.

The rainfall intensity, i, the rate at which water is applied to an area as rain, is distributed as shown in Fig. 13.

The run-off, Q, is divided into that portion which will reach the point of concentration in time to contribute to maximum flow and that which is

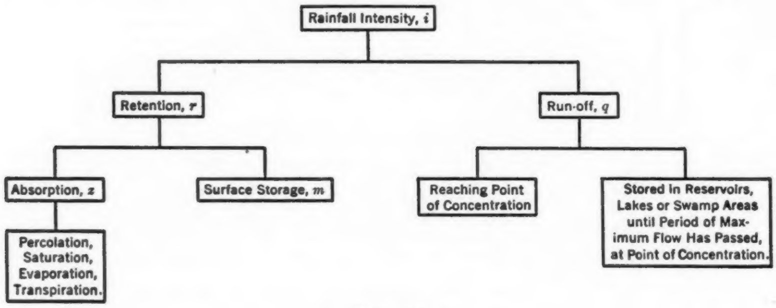

FIG. 13.

delayed, as storage in lakes, swamps, or reservoirs, until after the condition of maximum flow is past. The latter becomes a direct deduction from effective area.

The retention, r, is a factor embracing all other losses to run-off. It is made up of a factor, z, called absorption, and a retardation factor, m, which might be referred to as surface storage.

The factor, z, is a function of atmospheric conditions and rainfall intensities throughout the critical period of the storm and of the amount and rate of rainfall immediately preceding. It also reflects the capacity of the soil to pass water to underground storage, as percolation, or to retain it as absorption. The formula,

$$i = q + r \qquad (124)$$

is a straight-line relationship in which slope, and, therefore, the value of C, is unity. This is demonstrated by plotting i with q, and projecting the points with slope = 1, to the i-axis, where the intercepts will become the values for r.

If a point group is enveloped by a curve having a slope, M, and the points are projected with that slope to the i-axis, the intercepts, z, measuring absorption, will result; thus,

$$i = Mq + z \text{ ....} (125)$$

$$q = \frac{1}{M}(i - z) \text{ ....} (126)$$

and,

$$C = \frac{q}{i} = \frac{1}{M}\left(1 = \frac{z}{i}\right) \text{ ....} (127)$$

The factor, $\frac{1}{M}$, or $C_{max.}$, becomes constant for, and the maximum drainage coefficient on any water-shed is established by, its topography, type of soil, vegetable cover, and degree of development. This factor, modified by the combined effects of i and z, becomes the coefficient, C, for a particular storm.

The retardation factor, m, herein classified as surface storage, is represented by the difference between unity and $C_{max.}$ for any water-shed. It is a function of topography, shape, and development of the water-shed and is the factor, acting through surface storage as an equalizing influence, which establishes the rate at which the various parts of the water-shed will contribute to maximum flow; thus,

$$i = q + r = Mq + z \text{ ....} (128)$$

$$q = i - r = \frac{i - z}{M} \text{ ....} (129)$$

$$i\left(1 - \frac{1}{M}\right) = r - \frac{z}{M} \text{ ....} (130)$$

and,

$$1 - \frac{1}{M} = \frac{r}{i} - \frac{z}{Mi} \text{ ....} (131)$$

That m (or $1 - \frac{1}{M}$, or the ability of a water-shed to delay the delivery of run-off water to the point of concentration), is a function of the slope, M, and not a deduction factor, as z, is illustrated in the rainfall and run-off studies,[52] by Ivan E. Houk, M. Am. Soc. C. E. Bare soil, relatively incapable of retarding run-off is shown to have slope, M, very little different from 1, or to have a low value of $1 - \frac{1}{M}$, while the plots in which the soil was spaded and raked, representing a surface condition conducive to storage, showed a correspondingly high value of m.

Where water-sheds are closely grouped and where conditions are similar, a composite point group may be enveloped by a curve that will apply to the individual water-sheds, as shown in Figs. 14 and 15.

[52] Technical Reports, Miami Conservancy Dist., Pt. VIII, Fig. 24, p. 124.

Using values of P_1,

$$C_b = 0.284 \left(1 - \frac{z}{i}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (132)$$

and with P_2,

$$C_b = 0.287 \left(1 - \frac{z}{i}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (133)$$

Values for C_b, for Central Illinois, are listed in Columns (11) and (18) of Table 17.

Comparison is made with the rainfall-run-off studies[53] of C. E. Ramser, M. Am. Soc. C. E., in Western Tennessee. Values of $C_{\text{max.}}$ for the individual water-sheds are compared with the composite value in Fig. 15.

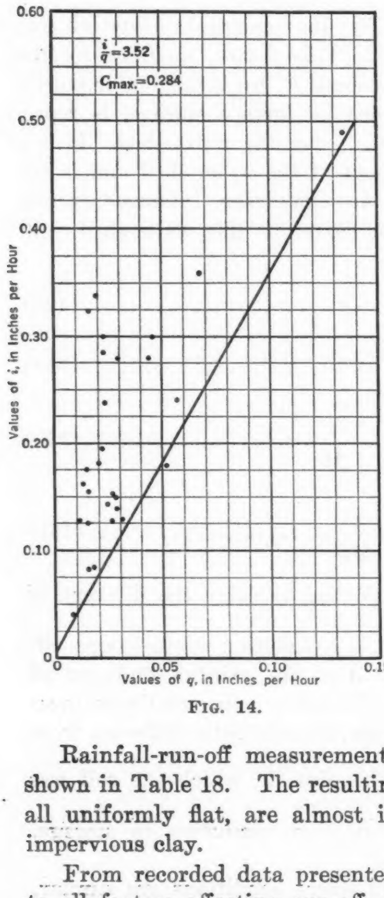

FIG. 14.

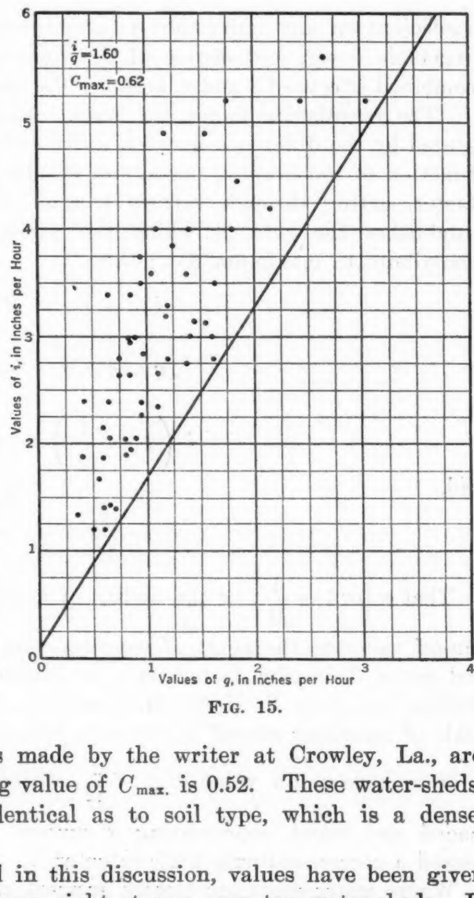

FIG. 15.

Rainfall-run-off measurements made by the writer at Crowley, La., are shown in Table 18. The resulting value of $C_{\text{max.}}$ is 0.52. These water-sheds, all uniformly flat, are almost identical as to soil type, which is a dense, impervious clay.

From recorded data presented in this discussion, values have been given to all factors affecting run-off, from eight storms over ten water-sheds. It has been shown that, from data made available by any reasonably accurate

[53] "Run-Off from Small Agricultural Areas," *Journal of Agricultural Research*, Vol. 34, No. 9, p. 799.

topographic survey (except the direct measurement of Kutter's n to be found in the Illinois data, which would ordinarily be assumed), all factors except z and $C_{max.}$ may be given determinate values. The writer has expressed Q in terms of storm frequency, but not flood frequency, and to that extent has fallen short of the goal of direct flood-flow determination.

The tendency in design will be to use $C_{max.}$ as the drainage coefficient, considering the margin between it and the values actually developed as a factor

TABLE 18.—Rainfall-Run-Off Measurements, at Crowley, La.

Water-Shed	Storm	t, min.	Max. q	i	z	C_a	C_b
H-4.......	1	730	0.0490	0.139	0.046	0.35	0.35
H-4.......	2	730	0.0390	0.102	0.028	0.38	0.38
H-5.......	1	880	0.0354	0.144	0.077	0.25	0.23
H-5.......	2	880	0.0340	0.090	0.025	0.38	0.38
H-6.......	3	1 050	0.0347	0.090	0.023	0.39	0.39
G-1.......	1	100	0.0460	0.090	0.003	0.51	0.51
G-2.......	1	380	0.0343	0.103	0.038	0.33	0.33
G-2.......	2	380	0.0295	0.104	0.048	0.28	0.28
G-2.......	3	380	0.0287	0.144	0.089	0.20	0.20
G-6.......	1	730	0.0640	0.138	0.016	0.46	0.46
G-6.......	3	730	0.0490	0.102	0.009	0.48	0.48

of safety. This suggests an analogy between the design of hydraulic structures to remove storm run-off and the design of structural members, in that $C_{max.}$ may be compared to the ultimate strength of a beam, the former being fixed by features of topography, soil, and cover of the water-shed, and the latter by its material, shape, and size. The actual C, resulting from any storm, may be compared to the working stresses developed in the beam, both occurring in any magnitude and with any frequency. The ratio of ultimate strength to working stress is taken as a factor of safety, as may the ratio of $C_{max.}$ to the actual C. Here the analogy ends, however, for the entire structure may depend upon the beam, while in the case of the drainage conduit or channel only nominal damage will result from overstressing its capacity.

It is not within the scope of this discussion to go into the problem further, except to suggest a possible approach to each of the factors remaining indeterminate.

The occurrence of a low value of z with a high value of i must be regarded as a purely fortuitous phenomenon, with the chances to combine being the product of their respective frequencies. If a flood-flow frequency of once in 5 years, using T_i and T_z equal to 5, is the design criterion, then the designer is, in reality, providing capacity, the chances to be reached or exceeded being once in 25 years. The factor, z, as in the case of i, can be expected to reduce as rainfall duration increases. It can also be expected to reduce as storm magnitude and duration, in terms of T_i, increase. If z and i, in terms of T_i and T_z, are combined in Equation (127) as $\sqrt{T_i}$ and $\sqrt{T_z}$, the resulting value of C, by reducing the needlessly large safety factor involved in the use of $C_{max.}$, will produce a more balanced ratio between efficiency and economy in design and will give values of Q having an approximate flood frequency, T_f. At least, the design will have taken cognizance of the fact that the extreme values of i and z combine infrequently.

It is reasonable to expect low values of z during or immediately following an extended wet spell, disregarding, of course, winter storms on frozen water-sheds. What constitutes (in terms of frequency, intensity, and duration) a storm period to produce maximum saturation is not known, but an approach to it may lie in the fact that the expression for rainfall intensity, for the Eastern United States, $i = \frac{K\,T^x}{t^e}$, may be extended to duration periods of thirty days.[54]

Apparently, the range in the value of $C_{max.}$, for topographic and soil conditions to be found on the ordinary agricultural water-shed, is from 0.28 to 0.65. Tentative deductions from unity, C, to be added as the influences of topography, soil, and cover combine—and given as a guide to values of $C_{max.}$ for agricultural water-sheds are, as follows:

Topography.—	
Flat land, with average slopes of 1 ft. to 3 ft. per mile....	0.30
Rolling land, with average slopes of 15 to 20 ft. per mile..	0.20
Hilly land, with average slopes of 150 to 250 ft. per mile..	0.10
Soil.—	
Tight, impervious clay....................................	0.10
Medium combinations of clay and loam................	0.20
Open, sandy loam..	0.40
Cover.—	
Cultivated lands	0.10
Woodland	0.20

CHARLES H. LEE,[55] M. AM. SOC. C. E. (by letter).[55a]—This paper represents a thorough attempt to analyze the rational method of estimating flood discharge, particularly as applied to unimproved drainage areas. The rational method was originally introduced as an improvement in estimating maximum storm run-off from small urban and suburban areas. As has been stated by the authors and pointed out by the writer,[56] it is also applicable to drainage areas of natural streams, where it has practical application in the design of waterway areas for stream-bed structures, such as culverts, bridges, spillways, overflow dams, etc. The authors have made a real contribution to the subject in their searching and logical analysis of the "detail method" of application. They have also presented some very suggestive material in Part II, but it is open to question whether as yet it is a step in advance to attempt to apply the rational method as a formula. The element of judgment is still present both in the application of the method as well as in the choice of basic values for a given drainage area. Under such conditions no formula can be devised that would be "fool proof." It is not believed that such a formula would be much of an improvement over the use of empirical formulas.

[54] "Formulas for Rainfall Intensities of Long Duration," by Merrill M. Bernard, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., October, 1931, p. 1278.

[55] Cons. Hydr. Engr., San Francisco, Calif.

[55a] Received by the Secretary February 29, 1932.

[56] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 433.

There is an important difference in the application of the rational method to the design of storm sewers or long storm drains and to stream-bed structures. To construct the former of a capacity sufficiently large to carry the maximum possible storm discharge would ordinarily be prohibitive in cost, and it is customary to design for maximum storms likely to occur once in an assumed period of years. In the case of bridges, culverts, spillways, etc., the relative cost of the waterway area is small compared to the cost of the entire structure, and it is usually economic to provide for the absolute maximum flood discharge. This fact suggests the importance of thorough consideration of factors other than rainfall intensity, which may contribute to the occurrence of a maximum when applying the rational method to unimproved drainage areas. The mention of a few instances may be in order as a part of the discussion of the authors' paper.

For small drainage areas in arid or semi-arid regions subject to local storms of great intensity known as cloudbursts, special consideration should be given to the value of C. Soils behave quite differently in their ability to absorb surface water, depending on their initial condition of moisture. This condition may be: (*a*) At saturation with complete filling of all voids; (*b*) at field capacity with partial filling of voids after all gravitational water has drained out; (*c*) at the point (wilting coefficient) where all moisture has been removed so that vegetation wilts; or (*d*) with all except hygroscopic moisture removed. In a dry arid region during the hot summer months the quantity of hygroscopic moisture is small and there may be a high degree of dessication of the soil to considerable depth.

For moisture conditions between field capacity and saturation and with good drainage, rainfall upon the surface is readily absorbed and passes downward through the soil column. For moisture content less than field capacity the soil retains added moisture until that capacity is reached, and progressive downward penetration occurs only as field capacity is exceeded in the soil column above. For highly dessicated soils this retarding action becomes a positive resistance to wetting, illustrated by the action of single raindrops falling upon a dusty road, or the addition of moisture to dry peat. A rainfall of great intensity, falling upon the surface of an extremely dry soil column, may thus encounter an actual resistance to absorption, producing immediate saturation at the surface and run-off, with dry soil below.

In addition to this surface tension phenomenon, there may also be resistance to penetration produced by trapped air in the soil. As the solid water represented by field capacity varies roughly from $\frac{1}{2}$ in. to 1 in. per ft. of sandy to silty loam soil, it is obvious that a highly dessicated soil column of several feet might maintain this condition for the duration of the storm, with a value of C of nearly 1.00. On the other hand, if initially at field capacity, the same column would immediately commence the downward transmission of water absorbed at the surface with a value of C nearer 0.50. These considerations are especially important in a cloudburst country, such as that traversed by several of the transcontinental railways, and the long water supply conduits common in the West. The writer has also noted their importance in the coast region of Central California, where early fall rains following an excess-

ively dry season have produced higher rates of flood run-off into storm sewers from small unimproved areas than storms of equal intensity occurring during the winter.

Among other conditions that may produce maximum flood flows exceeding those estimated by a routine application of the rational method may be mentioned frozen ground, warm rain falling upon snow, the effect of fire in destroying the vegetation mat, and the natural flow in the stream from a preceding storm at the beginning of a storm of maximum rainfall intensity. The authors considered the condition of saturation of the surface from an immediately preceding storm, but did not mention the possibility of there being appreciable flow already in the stream.[56a] The theoretical absolute maximum resulting from the latter condition is double the value computed, assuming that a second storm of equal intensity and duration followed immediately after the storm for which the computations were made. An actual instance for which the data were analyzed by the writer indicated a discharge from a small drainage area 80% greater for a second storm immediately following an earlier one of about the same intensity.[57]

These instances illustrate the importance of a broad consideration of all factors that may enter the problem where the rational method is being applied to estimate maximum flood flow as a basis for designing stream-bed structures.

[56a] *Proceedings*, Am. Soc. C. E., April, 1931, p. 598.
[57] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 439.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FINANCING STREET AND HIGHWAY IMPROVEMENTS

Discussion

BY SAMUEL ECKELS, M. AM. SOC. C. E.

SAMUEL ECKELS,[9] M. AM. SOC. C. E. (by letter).[9a]—This presentation of contemporary practice and opinion, representative of various governmental jurisdictions connected with the financing of street and highway improvements, is interesting and is particularly meritorious in pointing out quite clearly the need for factual studies before sound conclusions of great importance can be reached.

Until recently financing methods have been predicated upon the theory of localized use. Improvement of streets and highways has been considered to benefit adjacent properties, and, generally, within limits of ability to pay, such improvement has been considered largely chargeable to adjacent properties. At present (1932), there are some modifications of method recognizing changed traffic conditions, but traffic interest and acceptance of public responsibility for costs are naturally to be measured by the extent of public use, other than local, and, in this regard (particularly with respect to improvements other than on State highways), there is deficiency of reliable information.

The most conservative theories of financing public improvements may be stated as: (1) Financing responsibility coincident with property rights and authority; (2) public contribution from public general funds; and (3) public general funds from general tax.

(1) Financing Responsibility Coincident with Property Rights and Authority.—With careful location, extensive grading, construction of artificial surfaces, erection of structures, and intensive public use, private ownership of public right of way becomes a legal myth. Facility of motor-vehicle movement nullifies boundaries. In many cases, with respect to rural

NOTE.—The paper by R. W. Crum, M. Am. Soc. C. E., was published in August, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1931, by Messrs. A. P. Greensfelder and C. R. Thomas; and February, 1932, by J. C. Carpenter, M. Am. Soc. C. E.

[9] Chf. Engr., State Dept. of Highways, Harrisburg, Pa.

[9a] Received by the Secretary January 12, 1932.

roads and busy streets, there is overburdening of local responsibility, and adjustment by public contribution is under way.

(2) Public Contribution from Public General Funds.—Historically, public improvements have lagged behind demand; transportation facilities have been incommensurate with traffic requirements; and demands for public contribution, especially with reference to State highways, have been so great that an extraordinary increase of State revenues has been required.

(3) Public General Funds from General Tax.—The urgent demand for public contribution, together with opposition to an increase of a general tax levy, and the recognition of special benefits to the operation of motor vehicles, has led, throughout the country, to special levies on motor fuel and motor vehicles that are remarkable for lack of uniformity. Table 1 was arranged from data given in a *Bulletin* of the Motor Vehicle Conference Committee, issued in January, 1931.

TABLE 1.—SUMMARY OF MOTOR-VEHICLE TAX RATES

Item	Motor-Fuel Tax		Motor-Vehicle Levies on Private Passenger Cars	
	Rate, in dollars per gallon	Number of States	Basis of tax	Number of States
(1)	(2)	(3)	(4)	(5)
1	0.020	7*	Flat rate	2*
2	0.030	10	Value	3
3	0.035	1	Weight	20
4	0.040	17	Horse-power †	14
5	0.060	3	Flat rate and weight	2
6	.....	....	Value and weight	1
7	.....	....	Weight and horse-power.	5
8	.....	....	Value, weight, and horse-power	1

* Also, the District of Columbia.
† Including also cubic-inch displacement in one State.

Thirty-four States and the District of Columbia tax motor vehicles as personal property, in addition to charging annual fees, computed variously as indicated in Table 1, Columns (4) and (5).

Important Basic Principles.—Variations in the rate of gas taxes and registration fees are unfortunate, of course, particularly with reference to adjoining States, but the extent of the need of revenue and the degree of ability or willingness to pay are not uniform throughout the States. It would seem, however, that there might be some elimination of differences in the bases of computing license fees.

From the Symposium on "Equitable Distribution for Highway Purposes of Motor Vehicle Licenses and Gasoline Taxes," conducted by the Highway Division of the Society during 1929, Mr. Crum draws three principal deductions, as stated in the paper under "Important Basic Principles." The following comments apply to these deductions in the order given by the author.

Principle 1.—License fees are readily defensible in so far as they are designed to cover costs of necessary supervision, protection, and exclusive or special benefit. In excess of these indicated limits, license fees are dis-

criminatory, and are explainable only as the product of necessity or policy. The statement that motor vehicle users should pay a fee to cover "readiness to serve," is open to argument inasmuch as road construction constitutes a public improvement, as logically chargeable to public general funds as education, for instance. Considered in the light of expediency rather than equity, argument is avoided. There is apparent difference of opinion as to whether the license fee should be in lieu of other direct tax, and the answer to this question is a prerogative of the State legislatures. Where the motor vehicle is exempted from personal property assessment it would appear that the license fee to some extent should be based upon valuation. Other bases of computation of the license fee logically are horse-power and weight.

The collection of license fees computed on valuation, weight, and horse-power might appear difficult, but this idea has been used in North Dakota for some years and appears to be practicable. The component part of the registration fee based on valuation—when the motor vehicle is exempted from personal property assessment—is reasonably equivalent to the State average of personal property tax exemption. The horse-power and weight components of the fee are computed supposedly with reference to the highway surface requirements of motor vehicles. The influence of weight is of particular importance with reference to heavy wheel loads, but unfortunately there is a general lack of sufficient information in this regard. The graduated fee in accordance with horse-power is quite reasonable with reference to incurring maintenance costs and to ability of car owners to pay; but it might be injudicious in favoring under-powered freight carriers.

Traffic conditions as well as physical characteristics of surface should be studied in connection with the proportioning of license fees and the theory of the "readiness to serve" charge. Large or slow vehicles might not exceed the width capacity or the carrying capacity of a road surface, but still might interfere seriously with the usefulness of the road to the preponderant class of motor vehicles of moderate dimensions and moderate weight. The speeding up of large heavy vehicles would not make them entirely unobjectionable, especially in mountainous districts where they might get out of control.

Principle 2.—The fact that all the States and the District of Columbia collect gas tax is evidence of the popularity of this form for the production of revenue. Since gasoline consumption increases with weight of load and with speed of travel, it would seem that this tax might be depended upon for total revenue for road construction and maintenance in States where motor vehicles are not exempt from personal property assessment. From a practical standpoint the revenues from gas tax should not be set aside exclusively for maintenance because separate funds for separate purposes frequently hamper administration.

The argument that no part of the funds raised by motor-vehicle license fees or gasoline taxes should be diverted to any other use than the construction, maintenance, and control of highways, is an effort toward consistency in the application of the theory of special fund for special benefit development. This argument is more reasonable in States where the motor vehicle is taxed as personal property than in other States. Such theories, however, should be

considered with respect to necessity and expediency and with the recognition that there is less objection to the determination of levies or imposts on ability to pay than in consideration of ability to resist payment.

Determining the Road Users' Share of Costs.—The problem as to what share of the annual expenditures on the various classes of roads and streets should be borne by motor-vehicle users is unavoidably a problem of compromise. The determination of roadway costs, including cost of building and annual cost for administration, maintenance, and periodic replacement or renewal, simply shows a total that has been fixed by practical limits. What part of the costs should be borne by each of the various interests benefited, namely, the Federal Government, State Government, local community, adjacent property, and the user, is a problem of voluntary contribution and adjustment.

Under present conditions this country has a Federal Aid System, other State highways, county roads, township roads, and streets of incorporated municipalities. The designation of Federal Aid and State highways has very probably been judicious, but it has not been based on qualifications that furnish factors for accurate apportionment of costs. Briefly, the Federal Government has encouraged the development of correlated State highway systems by Federal aid apportioned among the States, one-third by area, one-third by population, and one-third in proportion to the total mileage of all highways at the time of the passage of the Act. The various States have relieved counties or townships as the case might be of responsibility for construction and maintenance of certain principal roads, and have set aside motor funds for State highway purposes. To some extent the Federal Aid highways enter the jurisdiction of municipalities, and to some extent motor funds have been distributed among counties, townships, and municipalities for highway purposes. The conditions involving the designation of Federal Aid highways and State highways and of the apportionment of Federal, and the distribution of State, motor funds are scarcely capable of analysis with respect to interests benefited, or to proper allocation.

Primary Roads.—It is established practice that the State (with Federal aid, as it becomes available) construct and maintain the principal rural roads and the State highway system, including the Federal Aid System. The discontinuance of State responsibility and Federal aid at municipal corporation lines has been considered impractical. The Federal Aid Act has recognized the importance of assisting in the development of streets in municipalities of small population and in the scantily developed sections of the more populous municipalities, and, in some cases, the burden of carrying large volumes of State highway traffic into and through boroughs and cities has been accepted in part as State responsibility. Future development of by-passes will not entirely eliminate the State highway significance of connecting streets.

There is force to Mr. Crum's statement that "the rate at which a system can be built will depend upon what the benefited interests can afford, * * *." This is especially true with reference to streets carrying through traffic with limited assistance of Federal Aid and State contribution.

Need for Co-Operation.—Viewing a highway system as a co-operative enterprise of the people of the State and considering the relative advantages of the "pay as you go" plan and the issuance of bonds, it may be stated that the use of bond funds has appeared desirable where the expected value of contemplated improvements has been greater than the anticipated total of annual costs and when rate of increase of annual revenue indicated future sufficiency for amortization of bonds, without impairing working-fund requirements. The present approach of motor vehicle registration to stabilized totals lessens regard for bond issues as a means of financing highway improvements. When there is recourse to the sale of bonds for financing highway improvement, however, the advantages of serial issues are important. A serial issue can be designed to develop equal annual costs, the loss by difference between interest rate on sinking fund and interest rate on bonds is overcome, and the responsibility for custody of the sinking fund is avoided.

Annual Roadway Costs.—On the basis of expenditure from current funds, actual road costs are chargeable to depreciation and maintenance. Actual annual road charges, then, may be stated as average annual depreciation plus average annual maintenance cost.

When the proceeds of bond issues are used for financing improvements, and in studies of costs where various rates of initial investment and depreciation are to be taken into account, annual interest on the investment is to be considered as well as depreciation charge and maintenance cost.

The formula mentioned by Mr. Crum as having been developed by a committee for the Highway Research Board differs slightly from the method that has been used in the Pennsylvania State Department of Highways for a number of years. Equation (1) charges simple interest on construction cost; it indicates perpetuity of initial investment, and capitalizes cost of periodic maintenance.

The Pennsylvania method charges off all investments in an estimated period of economic life and enters periodic maintenance cost as a simple annual average. The Pennsylvania formula uses as a factor the expression, $\frac{i}{(1+i)^n-1}+i$, values of which are directly obtainable from tabulations in handbooks of accounting and financing, known as "the annuity 1 will buy," or "the annual sinking fund which will accumulate to 1 at the end of n years" plus interest.

The Pennsylvania formula is as follows:

$$F = Gg + C'c + Dd + M + N \qquad (2)$$

in which, F is the average annual total charge; G is the grading and drainage cost; g is the annual sinking fund rate for the period of years in which the grading and drainage cost is to be charged off, plus interest; C' is the cost of a durable type of surface, or the salvage value anticipated in temporary or semi-durable type surface; c is the annual sinking fund rate for the years of economic life of the surface, plus interest; D is the re-surfacing cost, including initial surface treatment; d is the annual sinking fund rate

for the years of period preceding re-surfacing, plus interest; M is the average annual cost of general maintenance; and N is the average annual cost of periodic maintenance.

Knowledge of Costs Helpful.—A method tried for a short time by the Pennsylvania State Department of Highways to show annual road-mile and car-mile costs, made use of records of costs of construction and maintenance and estimates of depreciation or salvage value and anticipated further economic life of improvements. For each year the cost was computed as the depreciation for the year, plus interest on the depreciated value, plus maintenance costs. This method required two charts for each section of road: One for recording traffic, itemized maintenance costs, depreciation and interest, total annual cost, average annual cost per mile, and average car-mile cost, with expectation of adjustment for relative weighting of truck-mile costs; and the other for special use to show annual costs of pavement only, including a column for reporting traffic, current value of pavement, depreciation and interest, maintenance costs, annual pavement-mile costs, and average pavement costs per car-mile.

Source of Funds.—Explanation of absorption of costs of highway improvements by the Federal Government, the States, local communities, real estate, and road users, in terms of respective benefits, suggests the need of extensive studies of classification and of source and destination of traffic. It is to be remembered, however, that ratios of allotment of cost are not to be fixed definitely for long terms. The public serviceability and use of the principal roads of the nation at the present time (1932), for instance, are very different from what they were in 1916 when the first Federal Aid Road Act was approved, and the mapping of State highway systems is carried out with such flexibility of purpose and standard that classification of roads as to jurisdiction can only be considered a matter of expediency.

Secondary and Third-Class Roads.—The distinction between primary, secondary, and third-class roads by definition is difficult. In some cases, there might be classification of highways in a State such that the Federal Aid System was identical with the State Highway System. This might be described as the primary system; a recognized system of other roads under county supervision, eligible to certain State-aid benefit, might be termed secondary; while the remaining roads might be identified as third class. There might be still other cases of more extensive State highway systems, including secondary roads as well as primary roads, in the Federal Aid System. If there is any effort to classify highways on a basis of traffic counts, it is evident that there must be a sliding scale, because the distinction between primary and secondary roads in one State might be an absurdity in another. From a practical point of view classification of highways must be more elaborate than "primary, secondary, and third-class" roads; that is, in accordance with present conceptions, more than three traffic ratings are required. The principal consideration is in the question of roadside surface, width, and type, although there are questions of relocation and grading for reduction of curvature and gradient that cannot be settled without consideration of traffic classification.

A tentative road classification based on traffic is suggested, as follows:

Class	Traffic, in vehicles per day	Class	Traffic, in vehicles per day
1	More than 3 000	4	250 to 500
2	1 000 to 3 000	5	100 to 250
3	500 to 1 000	6	Less than 100

In addition to traffic, each class would be defined as to: Width, grades, alignment, sight distance, and depth of pavement.

City Streets.—The greatest difficulties in the problems of planning and financing street improvements are traffic congestion and relief. After giving some recognition to corporate limits and corporation rights, and assuming that by-pass routes are furnished for so-called through traffic, the street problem is largely localized, and the consideration of special benefits and users' costs by special license fees appears to be possible. The task is difficult, however, and requires careful investigation and consideration.

Needed Research.—The city street problem is acute in many cases and it is probably the most important field for investigation at the present time. In addition to other lines of investigation suggested by Mr. Crum, another of great importance is to determine the responsibility for carrying all classes of traffic at all times on all public roads. At present, it is generally recognized to be impracticable to build and maintain all the public roads to sustain maximum, generally permissible, loads in all seasons. Adverse conditions are snow, grades, curves, and surface bearing capacity. Where local use does not justify high type construction and all-year maintenance, and where local ability to pay does not permit it, the determination of facts of other than local use offers a wide field of study.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

DESIGN OF LARGE PIPE LINES

Discussion

BY MESSRS. H. C. BOARDMAN, R. L. TEMPLIN AND R. G. STURM, AND F. KNAPP.

H. C. BOARDMAN,[39] M. AM. SOC. C. E. (by letter).[39a]—As used by Mr. Schorer, the terms "exact theory," "exact stress condition," and "exact knowledge," are repugnant to the writer. "Exact theory" teases the mind for definition; "exact knowledge" of "exact stress conditions" is a goal constantly striven for, but seldom attained.

The author states that the governing design stresses in the shell of a pipe line are "derived," but his derivation consists essentially of a statement of the Bauersfeld differential Equations (1), (2), and (3), which are unfamiliar to American engineers, and their application to a simple beam with cylindrical section. The resulting equations for transverse shear and direct longitudinal stresses are shown to agree with those obtained by the ordinary theory of flexure applied to a simple, hollow, cylindrical beam. Is this, in the mind of the average reader, an indication of the validity of the Bauersfeld equations, or of the validity of the beam formulas? The transition to the case of multiple supports is made by the statement, "It can be shown that, in case of continuous pipe lines, the direct longitudinal stresses can also be derived from the theory of continuous beams." The implication is that the longitudinal stresses in continuous pipe lines have been derived by the use of the Bauersfeld equations alone, without the use of the theory of continuous beams. The writer is unable to make this derivation. Partial liquid loadings, to which the Bauersfeld equations are presumably not applicable, because the surface loads are discontinuous, are treated by a rather cursory reference to a paper[40] written by Mr. A. Frey Samsioe covering half-full liquid loading.

NOTE.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in September, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1931, by Messrs. L. J. Mensch and W. P. Roop; December, 1931, by Messrs. Johannes Skytte, Donald E. Larson, Raymond J. Roark, and F. W. Hanna; January, 1932, by Messrs. Paul Bauman and L. J. Larson; and February, 1932, by Messrs. C. M. Orr and Edward J. Bednarski.

[39] Research Engr., Chicago Bridge & Iron Works, Chicago, Ill.

[39a] Received by the Secretary December 11, 1931.

[40] "Die Spannungen in Einem auf Mehreren Stützen in gleicher, gegeseitiger Entfernung Aufgelegten und zur Hälfte mit Wasser Gefüllten Rohr," by A. Frey Samsioe, Stockholm, 1926.

The writer feels that the author would have accomplished his purpose more directly, and to the greater satisfaction of American engineers, if he had omitted Equations (2) and (3) and simply stated, as in effect he has, that:

(*a*) The shell of a cylindrical pipe line full of liquid, and kept essentially round at two or more supports, acts as a membrane free from bending stresses, except for a narrow zone near the supports.

(*b*) Under the conditions of Statement (*a*), the shearing stresses on a transverse section of the shell are necessarily tangential, and are distributed in accordance with the ordinary theory of flexure as applied to a hollow, cylindrical beam.

(*c*) Under the conditions of Statement (*a*), the longitudinal shell stresses may be obtained by an application of the ordinary beam formulas.

With Statement (*a*) granted, Equations (1), (4), (8), (11), and (12) follow directly from the equations of statics expressing the equilibrium of inward and outward radial forces on a differential element of the shell.

Still more convincing, in the writer's opinion, would have been an application of the well-known Mueller-Breslau Equations (31), (32), and (33), to an analysis of the dead load and the full liquid load stresses in a ring cut out of the pipe some distance from the supports by two transverse planes, 1 in. apart—the ring being held in equilibrium by tangential shearing forces distributed as in a hollow, cylindrical beam, by vertical dead weight forces, and by radial pressure forces. Such an analysis leads very simply to Equations (4) and (8), and to zero bending stresses.

Mr. Schorer has done a great service in calling attention to the fact that the shearing stresses on a ring between two transverse planes are tangential. Other investigators, the writer among them, have fallen into the error of considering only the vertical components of the shearing forces on the edges of the ring. A little consideration will disclose the fact that the shearing forces must be tangential to satisfy the laws of statics as applied to a differential arc of the ring.

It is unfortunate that the author treats so lightly the conditions of partial liquid loading. It is feared that many engineers who, like the writer, do not read German, will fail to obtain a translation of Mr. Samsioe's paper, and, as a result, will harbor a suspicion that the full liquid loading emphasized by the author is not critical. For this reason the following analysis of ring stresses in a pipe half full of liquid is offered in the hope that, in combination with a test to be discussed later, it will help others, as it has helped the writer, to gain a clearer idea of the true stress conditions. The notation is essentially that of the author; departures therefrom are self-explanatory.

The ring chosen is 1 in. wide longitudinally, and the transverse plane of one edge is midway between two pipe supports. The shearing stresses are first assumed to be distributed in accordance with the ordinary theory of flexure as applied to hollow, cylindrical beams, their intensities being one-half those for full liquid loading. Later, it is shown that a different shear distribution would result in practically zero moments throughout the ring,

a condition which seems to correspond with the observed behavior of an experimental pipe section subsequently described.

In Fig. 23, $p = 0$ above the liquid, and below the liquid surface, $p = - q\, r \cos v$. Furthermore, $s = 0.5\ q\ r \sin v$, which is one-half the value of s for full liquid load. The forces, s and p, represent the external load system, which is symmetrical about the vertical diameter. For this reason, Y_1 is

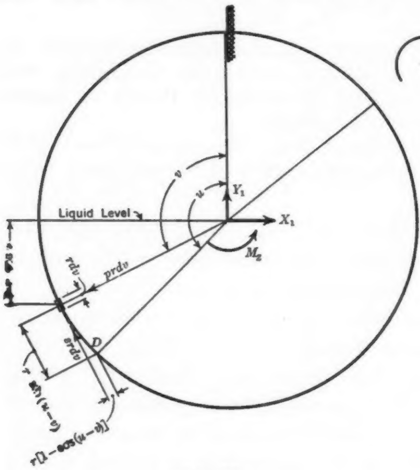

FIG. 23

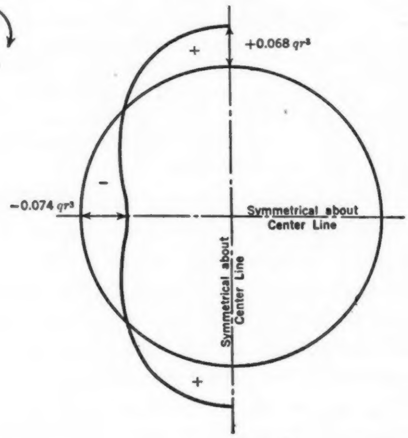

FIG. 24.

zero. The moment at the point, D, caused by the tangential shearing force on the arc, rdv, is, by inspection of Fig. 23:

$$M_D = s\ r\ dv \left\{ r\,[1 - \cos\,(u - v)] \right\}$$

which, by substitution of $0.5\ q\ r \sin v$ for s, becomes,

$$M_D = 0.5\, q\, r^2 \sin v\, dv \left\{ r\,[1 - \cos\,(u - v)] \right\}$$

or,

$$M_D = 0.5\ q\ r^3\ (\sin\ v - \cos\ u\ \sin\ v\ \cos\ v - \sin\ u\ \sin^2\ v)\ dv$$

In all quadrants the moment due to shearing forces, therefore, is,

$$M = 0.5\, q\, r^3 \int_0^u (\sin\ v - \cos\ u\ \sin\ v\ \cos\ v - \sin\ u\ \sin^2\ v)\ dv \ldots\ldots (147)$$

which by integration becomes,

$$M = 0.5\ q\ r^3\ (1 - \cos\ u - \tfrac{1}{2}\ u\ \sin\ u) \ldots\ldots\ldots\ldots (148)$$

There are no moments in the first quadrant due to the radial pressures, p. In the second and third quadrants the moment about D of the radial pressure forces acting on the arc, rdv, is, by inspection,

$$- p\ r\ dv \left\{ r \sin\,(u - v) \right\}$$

which, by substitution of $- q\ r\ \cos\ v$ for p, becomes,

$$M_D = q\ r^3\ \cos\ v\ dv\ \sin\ (u - v) = q\ r^3\ (\sin\ u\ \cos^2\ v - \cos\ u\ \sin\ v\ \cos\ v)\ dv$$

In the second and third quadrants the moment due to pressure forces, therefore, is $q\,r^3 \int_{0.5\pi}^{u} (\sin u \cos^2 v - \cos u \sin v \cos v)\, dv$; or,

$$M = 0.5\ q\ r^3\ (u \sin u - 0.5\pi \sin u + \cos u) \qquad (149)$$

By inspection, the moment in the fourth quadrant due to the liquid pressures is,

$$M = (0.5\ q\,\pi\,r^2)\ r \sin u = 0.5\ q\,\pi\,r^3 \sin u \qquad (150)$$

Summarizing: In the first quadrant,

$$M_{e1} = 0.5\ q\ r^3\ (1 - \cos u - 0.5\ u \sin u) \qquad (151)$$

in the second and third quadrants,

$$M_{e23} = 0.5\ q\ r^3\ (1 - \cos u - 0.5\ u \sin u) + 0.5\ q\ r^3\ (u \sin u - 0.5\,\pi \sin u + \cos u) \qquad (152)$$

and, in the fourth quadrant,

$$M_{e4} = 0.5\ q\ r^3\ (1 - \cos u - 0.5\ u \sin u) + 0.5\ q\ r^3\ (\pi \sin u) \qquad (153)$$

Equation (34) then becomes,

$$X_1 = \frac{1}{\pi\,r}\left[\int_0^{\frac{\pi}{2}} M_{e1} \cos u\, du + \int_{\frac{\pi}{2}}^{\frac{3\pi}{2}} M_{e23} \cos u\, du + \int_{\frac{3\pi}{2}}^{2\pi} M_{e4} \cos u\, du\right] \qquad (154)$$

from which,

$$X_1 = -\ \tfrac{1}{4}\ q\ r^2 \qquad (154)$$

Equation (36) becomes,

$$M_Z = -\frac{1}{2\,\pi}\left[\int_0^{\frac{\pi}{2}} M_{e1}\, du + \int_{\frac{\pi}{2}}^{\frac{3\pi}{2}} M_{e23}\, du + \int_{\frac{3\pi}{2}}^{2\pi} M_{e4}\, du\right] \qquad (155)$$

from which,

$$M_Z = q\,r^3\left(\frac{1}{\pi} - \frac{1}{2}\right) \qquad (156)$$

By combining M_z and the moments due to X_1, with the M_e moments expressed by Equations (151), (152), and (153), the following ring moments are obtained,

$$M_1 = q\ r^3\left(\frac{1}{\pi} - \frac{1}{4}\cos u - \frac{1}{4}\,u \sin u\right) \qquad (157)$$

$$M_2 = M_3 = q\ r^3\left(\frac{1}{\pi} + \frac{1}{4}\cos u - \frac{\pi}{4}\sin u + \frac{1}{4}\,u \sin u\right) \qquad (158)$$

and,

$$M_4 = q\ r^3\left(\frac{1}{\pi} - \frac{1}{4}\cos u + \frac{\pi}{2}\sin u - \frac{1}{4}\,u \sin u\right) \qquad (159)$$

A study of the expressions for Equations (157), (158), and (159) will show that the moment curves are symmetrical about both the horizontal and vertical axes (see Fig. 24).

Attention is called to the fact that the moments in Fig. 24 are those which would exist if the transverse shears were distributed in accordance with the ordinary theory of flexure as applied to hollow, cylindrical beams. They are of considerable magnitude; for example, if applied to the pipe line chosen by the author to illustrate the application of his formulas, the moment of 0.074 $q\ r^3$ for a half-full water load would result in a calculated bending stress of $\dfrac{0.074 \times 0.362 \times 60^3}{\frac{1}{6} \times (\frac{1}{4})^2} = 55\,000$ lb. per sq. in., indicating large deformations and over-stress while the pipe is being filled.

The writer believes no such over-stress or deformation would actually occur. He has recently directed the testing of a copper cylinder, $22\frac{1}{2}$ in.

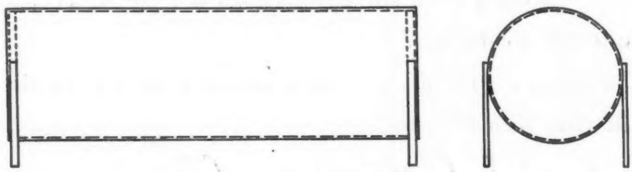

FIG. 25.

in diameter and 68 in. long, made of No. 24 gauge material. A wooden disk was closely fitted into each end and the cylinder was supported on four vertical wooden legs, two nailed to each disk, as shown in Fig. 25.

The cylinder was slowly filled with water, and measurements were made at the mid-section to determine the changes in shape. The recorded variations were very small, and not perceptible to the eye, although the calculated bending stress due to a moment of 0.074 $q\ r^3$ is $\dfrac{0.074 \times 0.0362 \times 12.25^3}{\frac{1}{6} \times (\frac{1}{40})^2}$ $= 36\,000$ lb. per sq. in., probably more than the ultimate strength of the copper. As a check the filling and measuring were done three times, always with essentially the same result. The ratio of diameter to thickness was 900. The shells of large oil-storage tanks show a similar remarkable resistance to deformation under wind loads, if they are kept round at the top by suitable circular girders.

Only one explanation seems possible. The transverse shears must be distributed so as to result in practically no moments in rings cut out by transverse sections. A clue to the approximate magnitude and direction of the shearing forces which must be applied to a ring acted upon by shearing stresses distributed in accordance with the ordinary theory of flexure, in order to result in zero moments, may be gained by an inspection of Fig. 24. The moment graph is the general shape the ring would take if the indicated moments actually existed. It is apparent that four tangential forces acting as shown in Fig. 26 would tend to pull the ring into its original circular shape.

Assuming each of these forces to be unity, the resulting moments through the ring can be found as follows, using the Mueller-Breslau equations. Since the load system is symmetrical about the vertical diameter, Y_1 is zero. The moment of the force at A about any point, P, if $u > \frac{\pi}{4}$, is, by inspection, $-\left[r\left\{1-\cos\left(u-\frac{\pi}{4}\right)\right\}\right]$, or,

$$M = -1 + 0.707 \cos u - 0.707 \sin u \;\ldots\ldots\ldots\ldots (160)$$

Similarly, the moment of the force at B about any point P, if $u > \frac{3\pi}{4}$, is,

$$M = 1 + 0.707 \cos u - 0.707 \sin u \;\ldots\ldots\ldots\ldots (161)$$

the moment of the force at C about any point, P, if $u > \frac{5\pi}{4}$, is,

$$M = 1 - 0.707 \cos u - 0.707 \sin u \ldots\ldots\ldots\ldots (162)$$

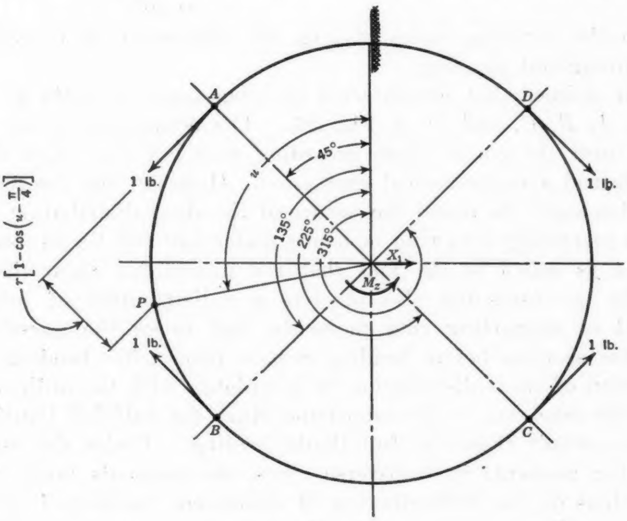

FIG. 26.

and the moment of the force at D about any point, P, if $u > \frac{7\pi}{4}$, is,

$$M = 1 - 0.707 \cos u + 0.707 \sin u \ldots\ldots\ldots\ldots (163)$$

Combining: For values of u (Fig. 26), between 0° and 45° and 315° and 360°,

$$M_e = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (164)$$

for values of u between 45° and 135°,

$$M_{ea} = -1 + 0.707 \cos u + 0.707 \sin u \ldots\ldots\ldots\ldots (165)$$

for values of u between 135° and 225°,

$$M_{eb} = 1.414 \cos u \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (166)$$

and, for values of u between 225° and 315°,

$$M_{ec} = -1 + 0.707 \cos u - 0.707 \sin u \ldots\ldots\ldots (167)$$

By inspection, $X_1 = 0.707$ and Equation (36) becomes,

$$M_Z = -\frac{1}{2\pi}\left[\int_{\frac{\pi}{4}}^{\frac{3\pi}{4}} M_{ea}\,du + \int_{\frac{3\pi}{4}}^{\frac{5\pi}{4}} M_{eb}\,du + \int_{\frac{5\pi}{4}}^{\frac{7\pi}{4}} M_{ec}\,du\right] \ldots (168)$$

from which, $M_z = 0.5\ r$.

By combining M_z in Equation (168), and the moments due to X_1 with the moments given by Equations (165), (166), and (167), ring moments of -0.207, $+0.207$, -0.207, and $+0.207$, are obtained for points, $u = 0°$, $u = 90°$, $u = 180°$, and $u = 270°$, respectively. The corresponding moments of Fig. 24 are, $0.068\ q\ r^3$, $-0.074\ q\ r^3$, $+\ 0.068\ q\ r^3$, and $-\ 0.074\ q\ r^3$, respectively, for which the average arithmetical value is $0.071\ p\ r^3$. It follows that a shearing force at *A, B, C,* and *D*, of $\frac{0.071}{0.207}\ q\ r^3$, or $0.343\ q\ r^3$, in addition to the shearing forces of Fig. 23, will result in practically zero moments throughout the ring.

It is not claimed that concentrated shearing forces of $0.343\ q\ r^3$ exist at the points, *A, B, C,* and *D* of Fig. 26. The actual forces are doubtless distributed over the entire circle according to a law for which the writer has not obtained a mathematical expression. However, the foregoing crude demonstration serves to reveal the nature of the shear distribution necessary to result in practically zero ring moments under half-full liquid loading.

Attention is called to the fact that the unorthodox shear distribution suggested in the foregoing also involves a redistribution of longitudinal stresses and of supporting ring moments, and raises the question as to whether plane sections before bending remain plane after bending. Under the assumption of shear distribution in accordance with the ordinary theory of flexure, the moments in the supporting rings for half-full liquid loading are exactly one-half those for full liquid loading. Under the assumption of no bending moments in transverse rings, the moments taken from the transverse rings by the redistribution of shears are transferred to the supporting rings. The resulting supporting ring moments at $u = 0°$ and $u = 90°$, for half-full liquid loading, are $+\ 0.0546\ L\ q\ r^3$ and $-\ 0.0897\ L\ q\ r^3$, respectively, as compared with $-\ 0.0267\ L\ q\ r^3$ and $\pm\ 0.0314\ L\ q\ r^3$, for full liquid loading. The half-full liquid moments are obtained by combining 50% of the moments of the author's Fig. 8, with L times the moments of Fig. 24; the full liquid moments are those of the author's Fig. 8. These results indicate that the maximum moment in the supporting ring for half-full liquid loading is approximately three times the maximum moment for full liquid loading. The writer has arrived at this conclusion by a combination of observation and logic. If his reasoning is faulty, he will appreciate correction.

Mr. Schorer is to be congratulated on his effort to rationalize pipe design. His paper is instructive and stimulating. However, he has thrown himself open to the charge of concentrating on the condition of loading for which the stress analysis is simplest, namely, full liquid loading. For this reason his paper should be supplemented by further investigation, both analytical and experimental, to establish, conclusively, the stress conditions in supporting rings and cylinder for all partial liquid loadings.

R. L. TEMPLIN,[41] M. AM. SOC. C. E., AND R. G. STURM,[42] ASSOC. M. AM. SOC. C. E. (by letter).[42a]—The author is to be commended for bringing before the Engineering Profession so vital a subject. His paper presents one method of supporting a thin-walled cylinder so that the stresses set up in the shell are not serious even when spans greater than those in common practice are used. This, however, should not be construed as the only method of supporting a pipe to obtain small distortions and low stresses.

The writers agree with the fundamental principles and basic assumptions set forth, but feel that several questions which have a direct bearing on the usefulness of the theory might well be clarified. The first question is in regard to the eccentricity, a, of the reaction which is the first term defined in the "Notation." Referring to Fig. 6 and the subsequent analysis, it seems that the distance, a, should be defined as measured from the neutral axis of the ring. The location of the neutral axis of the ring and shell requires some knowledge or assumptions as to how much of the shell acts with the stiffener, because the pipe shell itself will tend to act integrally with the ring to a certain degree. In the illustrative example given by the author, the variation in the location of the neutral axis may be more than 100% of the computed eccentricity of reaction. As no measured values are given of stresses that actually exist in a pipe so stiffened and so supported, deviations of the theoretical stresses from the actual stresses are not known.

It is interesting to note that the two radii, r and R, both occur in the first set-up of the equations, but the final formulas contain only R, thereby eliminating the effect of the relative size of stiffener and pipe. The writers believe that neglecting this effect is a justifiable approximation in the practical cases, although it does not give an exact theory.

Another question which may be raised is: "How does the example of a pipe, supported as in the case of the Glines Canyon pipe, prove or disprove the theory when it does not appear to comply with the assumptions made in the analysis?" In the first place, the supports of the Glines Canyon pipe are riveted securely to the stiffener ring over a considerable distance extending from the horizontal diameter to a point well below it. Therefore, the point of support is not definitely at the ends of the horizontal diameter nor at any fixed point with reference to the center line of the girder; nor is it supported

[41] Chf. Engr. of Tests, Aluminum Research Laboratories, Aluminum Co. of America, New Kensington, Pa.

[42] Research Engr. Physicist, Aluminum Research Laboratories, Aluminum Co. of America, New Kensington, Pa.

[42a] Received by the Secretary January 9, 1932.

on knife-edges as assumed in the analysis of the paper. The writers would like to ask, therefore, "Does the fact that this pipe stands without appreciable distortion or without developing serious stresses, necessarily indicate the validity of the analysis presented in the paper?" Apparently, it does show that a pipe reinforced by stiffeners and supported on strut supports has appreciable advantages over pipe supported by other means.

Early in 1928, one of the writers was called upon to make extensive field tests of the Santeetlah Pipe Line which was part of one of the hydro-electric power developments of the Tallassee Power Company in Western North Carolina. This line, including tunnels, is about 5 miles long, and was fabricated from steel plates varying in thickness from $\frac{3}{8}$ in. to $\frac{1}{2}$ in. so as to form a circular conduit having an inside diameter of 11 ft. The pipe is supported on saddles at intervals of about 30 ft. These saddles were fabricated from steel plates and angles so as to extend over an arc of 90° of the pipe shell, and, in turn, are supported on concrete piers or steel trestle bents, depending on the topography. The pipe was stiffened at the supports with 4 by 6 by $\frac{3}{8}$-in. angles riveted to the shell circumferentially, with the longer leg outstanding. The saddles were spaced so as to be adjacent to or framed into one of the stiffening rings.

Tests were made to determine the change in the stresses at critical points in the pipe shell when it was subjected to various water loads up to the maximum load. The places at which the stresses were measured included various points on the stiffening rings, stresses in different directions in the pipe shell around the ends of the saddles and adjacent to the stiffeners, and at various points around the entire circumference of the pipe at mid-span and near the supports. The deformation of the shell from the initial circular section was also determined.

The results of these first field tests showed that when the pipe was almost full of water high stresses occurred in both the saddles and the stiffening rings at certain critical points, namely, in the shell near the ends of the saddles, in the saddle stiffener rings at the top of the vertical diameter, and at both ends of the horizontal diameter. The amount of stiffening ring steel was increased, therefore, by arc welding additional angles to those already on the pipe so as to form a **Z**-section. These added angles were 4 by 4 in. by $\frac{3}{8}$ in., and were attached to the others so that the final depth of the stiffening ring was about 7 in.

After the completion of this reinforcing work, the pipe was again tested with more extensive measurements than previously. These later tests showed that the high stresses found in the first tests, were decreased substantially to acceptable safe working values. Strengthening the stiffener rings automatically relieved the high stresses in the saddles and in the pipe shell.

Although this pipe line had been designed according to a theory that gave satisfactory results on smaller sizes yet deficiencies in the design theory became apparent when it was applied to lines of the size of that just described. Accordingly, an extensive research was carried out on the design of pipe lines and their supports. This research included tests of models, together with a development of a theory of design which differs from that given by Mr. Schorer.

The models were built to approximately one-tenth the scale of the Santeetlah Pipe Line, and were made of aluminum alloy sheet, so as to magnify the strains and distortions. The models were supported on saddle supports which were in contact with the pipe shell over distances of 66°, 90°, and 180° of arc. These saddles were arranged so as to give a series of spans varying from 3 to 15 diameters, the total length of the model pipe being 15 diameters.

Since models of such size preclude any extensive measurement of localized strains it was decided to depend for the most part on the deformation or distortion of the pipe shell as an index of such strains and hence stresses resulting from various water loads. Strains were actually determined, however, with ½-in. "tensometers," at some points in the models.

Fig. 27 shows a sketch of one of the models in position for test with supports at intervals of 3 diameters. Fig. 28 shows typical distortion curves for an unstiffened pipe on three kinds of saddles. The maximum distortion was found to occur when the pipe was just full of water, but under no pressure.

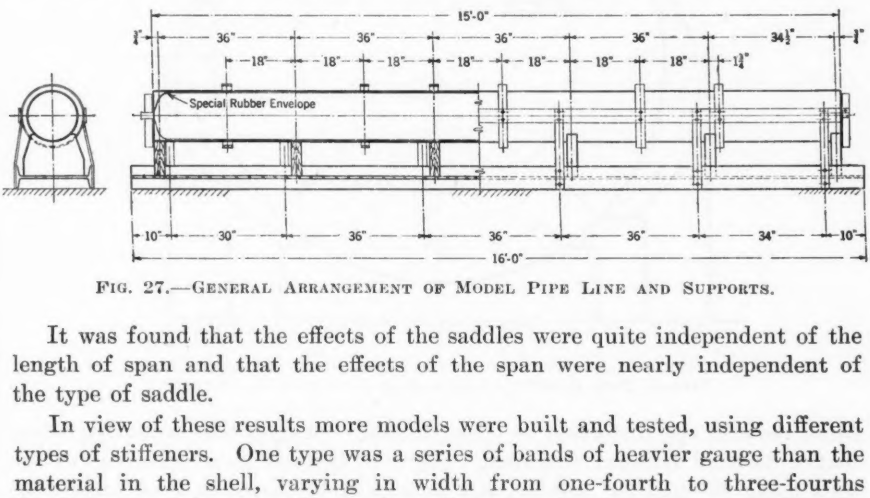

FIG. 27.—GENERAL ARRANGEMENT OF MODEL PIPE LINE AND SUPPORTS.

It was found that the effects of the saddles were quite independent of the length of span and that the effects of the span were nearly independent of the type of saddle.

In view of these results more models were built and tested, using different types of stiffeners. One type was a series of bands of heavier gauge than the material in the shell, varying in width from one-fourth to three-fourths the diameter of the pipe. The main shell was cut for the space occupied by the bands, except for the metal required to make the lap-joints. The bands were bolted to the pipe with small machine screws (No. 2), spaced approximately 1 in. on centers.

Another type of stiffener was made up of two channels, which were placed back to back at the joints of the pipe. The pipe was arranged so that the saddles came adjacent to one side of the stiffener rings thus formed. The types of saddles and spans were the same as before. Stresses were measured with "tensometers" attached to the stiffeners as shown subsequently in Fig. 33.

It was found that the distortion of the pipes, stiffened with the bands at the supports, was very little less than the distortion of the unstiffened pipe, and was found to vary over wide limits because of the indefinite connection between the bands and the shell by means of the bolts used. However, in cases where the bolts apparently held, the distortion varied inversely with the

moment of inertia of the combined shell and band. The points of distress and points of contraflexure were found to be the same as in the prototype, and the points of distress as determined by stress measurements on the stiffeners showed that the variation of stresses in the model was in good agreement with the variation of those measured in the Santeetlah Pipe Line. It was also found that the stiffeners made up of two small channels back to back were much more effective in reducing distortions than the bands. One-fifth as much added material in the form of a stiffener ring rather than a heavy

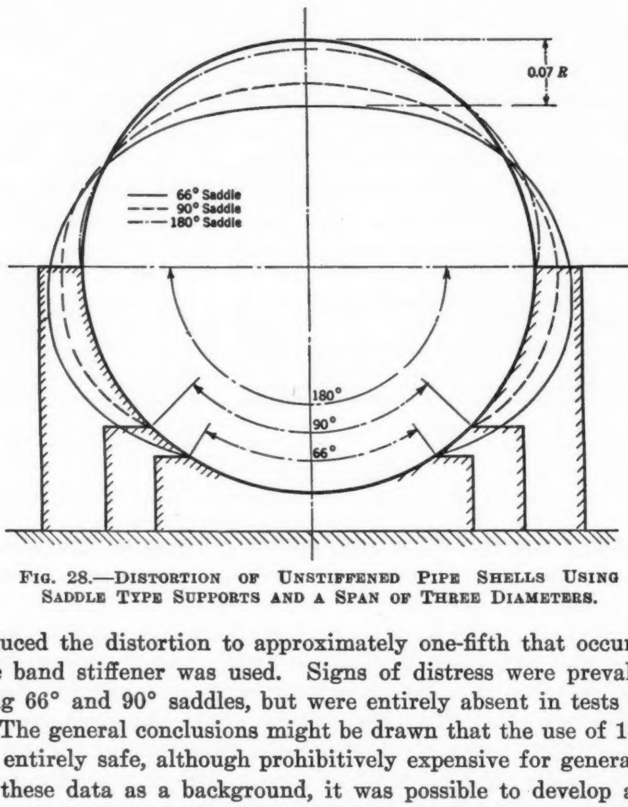

FIG. 28.—DISTORTION OF UNSTIFFENED PIPE SHELLS USING SADDLE TYPE SUPPORTS AND A SPAN OF THREE DIAMETERS.

band, reduced the distortion to approximately one-fifth that occurring when that wide band stiffener was used. Signs of distress were prevalent in the tests using 66° and 90° saddles, but were entirely absent in tests using 180° saddles. The general conclusions might be drawn that the use of 180° saddles would be entirely safe, although prohibitively expensive for general use.

With these data as a background, it was possible to develop a theory of the action of a horizontal pipe line, just filled with liquid, supported on saddles which would give reliable indications of stresses and distortions of stiffened or unstiffened pipe shells supported on various types of saddles. The general differential equation expressing the equilibrium of any elemental section of the pipe shell was derived from a consideration of a free body diagram of such a small portion. Fig. 29 is a sketch of a small element of a pipe shell showing all the forces acting on that small part. From a consideration of equilibrium of such a small particle, a general differential equation involving the forces acting on such an element is obtained. Then, assuming that, since the shell is thin, ordinary flexural formulas hold, the equations of continuity may be combined with the differential equation of equilibrium so as

to give a general differential equation in terms of the distortion, ρ, of the shell. This equation was found to be:

$$\frac{EI}{R^4(1-m^2)}\left[\frac{\partial^4 \rho}{\partial u^4} + R^2 \frac{\partial^2 \rho}{\partial u^2} + 2R^2 \frac{\partial^4 \rho}{\partial u^2 \partial (x')^2} + m R^4 \frac{\partial^2 \rho}{\partial (x')^2} + R^4 \frac{\partial^4 \rho}{\partial (x')^4}\right]$$

$$= -N' - \frac{T_2}{R_2}\left(R - \frac{\partial^2 \rho}{\partial u^2}\right) - T_1 \frac{\partial^2 \rho}{\partial (x')^2} \quad \ldots\ldots\ldots\ldots (169)$$

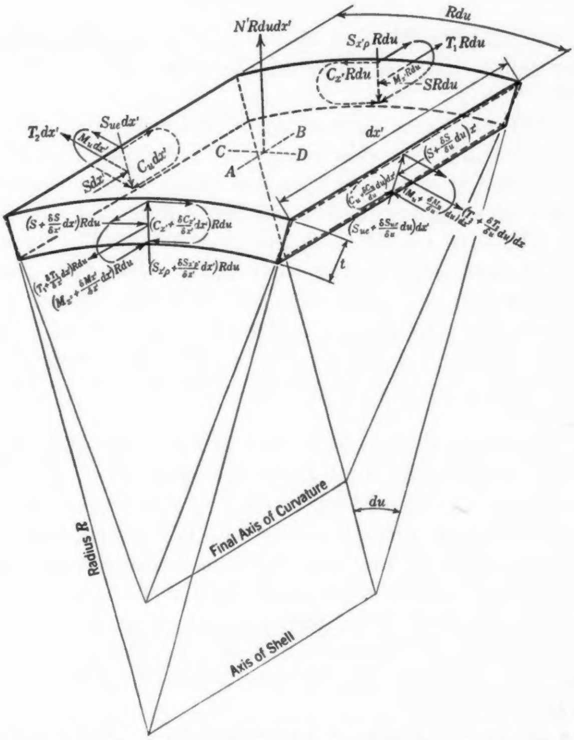

FIG. 29.—FREE BODY DIAGRAM FOR AN ELEMENT OF A THIN CYLINDRICAL SHELL

in which,

x' = distance along the pipe from the center line to the point considered;
ρ = change in radius at any point;
R = mean radius of the pipe shell;
N' = intensity of the hydrostatic pressure at any point;
E = modulus of elasticity of the material in the shell;
$I = \frac{t^3}{12}$ = moment of inertia of a cross-section of the shell of unit length;
t = thickness of the shell;
m = Poisson's ratio for the material in the shell;
T_2 = tangential pull, in pounds per linear inch, circumferentially;
T_1 = tangential pull, in pounds per linear inch, longitudinally.

In connection with Fig. 29 it should be noted also that ρ is positive outward; x' is positive toward the reader; u is positive clockwise; and, M is positive when it tends to cause tension in the outer surface.

The general solution of Equation (169) involves work very similar to that carried through in laborious detail by Mr. A. Frey Somsioe.[43] In view of the experimental data, certain simplifying assumptions were made with the result that the following equation was derived:

$$S = - qRx' \sin u \ldots\ldots\ldots\ldots\ldots\ldots (170)$$

in which, q = weight per cubic foot of the liquid in the pipe and u = angular position at any point. The meaning of the other terms are as defined for Equation (169).

Equation (170) has been derived by a different line of reasoning by D. Thoma.[44] In passing, it would be well to point out that in the mathematical work involved in deriving Equation (170) it was found that, of all the possible types of solutions for the general differential equation, a series of the form, $\rho = A_0 + A^2 \cos 2\theta + A_4 \cos 4\theta$ * * *, fulfills the conditions if A_0, A_2, and A_4 are considered as functions of x'. Considering only the first two terms of the series, one would conclude that the general shape of the distorted pipe with the ends fixed would tend to be that of a cosine curve. This would mean that the distortions of a shell with perfectly fixed ends would be of the same general shape as the measured distortions of the shell tested (see Fig. 28).

In the models and in the practical case of the Santeetlah Pipe Line the distortions at the support are very large compared with those of a "fixed ended" shell. Hence, the distortions at the supports were considered as the controlling factor. Using Equation (170) as the distribution of shearing forces on the cross-section of the shell, it seems reasonable that at the point of support a stiffening ring or some other rigid member should be attached to the pipe shell to transmit these forces to the support. The actual conditions may be quite closely approximated by considering that these forces are transmitted to a stiffening ring by the shell at the junction between the ring and the shell.

Fig. 30 is a schematic sketch of a portion of the shell near the support showing the forces acting on it. Treating this part of the structure as an isolated continuous ring with these forces acting on it, equations for the bending moments in the ring resulting from water load are found to be as follows:

For the 66° saddle:

$$M_u = qR^3L \left[0.6327 - 0.5046 \cos u - \frac{u}{2} \sin u\right] \ldots\ldots (171)$$

For the 90° saddle:

$$M_u = qR^3L \left[0.6791 - 0.5847 \cos u - \frac{u}{2} \sin u\right] \ldots\ldots (172)$$

[43] "Die Spannungen in einem auf mehreren Stützen in gleicher gegenseitiger Entfernung aufgelegten und zur Hälfte mit Wasser gefüllten Rohr," by A. Frey Samsioe, Ingeniörs Vetenskaps Akademien, Handlingar Nr. 50, 1926.

[44] *Zeitschrift* für das Gesamte Turbinenwesen, February 20, 1920.

For the 180° saddle:

$$M_u = qR^3L\,[0.8411 - 0.8211 \cos u - \frac{u}{2} \sin u] \ldots\ldots\ldots (173)$$

From the theory of elasticity of curved members involving the relationship,

$$M_u = \frac{EI}{R^2}\left(\frac{\partial^2 \rho}{\partial u^2} + \rho\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (174)$$

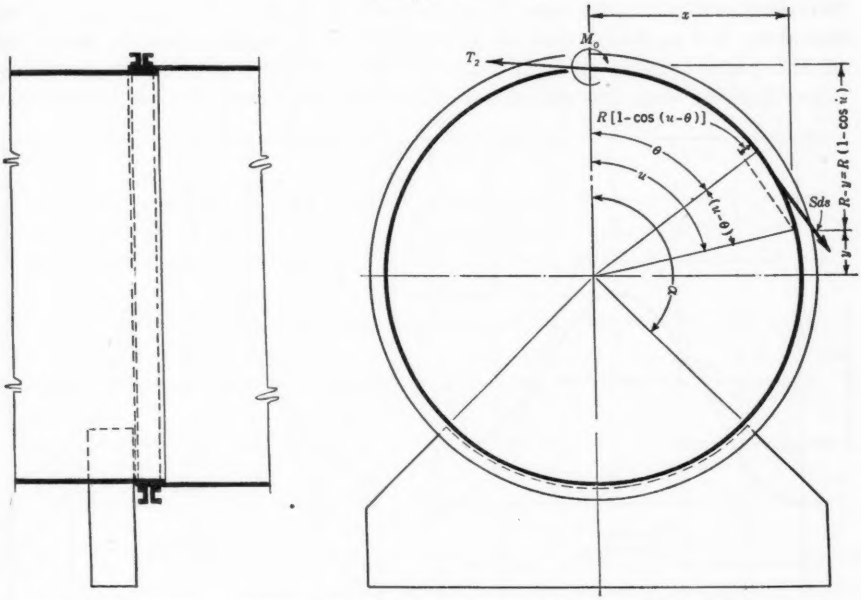

FIG. 30.—SCHEMATIC SKETCH OF PIPE AT SUPPORT, SHOWING FORCES ACTING ON IT.

the radial distortions may be found to be:

For the 66° saddle ($\alpha = 147°$):

$$\rho = \frac{q\,R^5\,L}{EI}\,[0.6327 - 0.6976 \cos u - 0.3773\,u \sin u + 0.125\,u^2 \cos u]\,.(175)$$

For the 90° saddle ($\alpha = 135°$):

$$\rho = \frac{q\,R^5\,L}{EI}\,[0.6791 - 0.7173 \cos u - 0.4173\,u \sin u + 0.125\,u^2 \cos u]\,..(176)$$

For the 180° saddle ($\alpha = 90°$):

$$\rho = \frac{q\,R^5\,L}{EI}\,[0.8411 - 0.8441 \cos u - 0.5355\,u \sin u + 0.125\,u^2 \cos u]\,..(177)$$

A comparison between the theoretical distortions and the distortions measured on the model involves a consideration of several important factors: First, since the stiffeners were bolted to the pipe shells there is a possibility of as much as 10% variation in the combined moment of inertia of the stiffener and that part of the plate acting with it. A difference between the

measured stresses at the same water levels in the prototype during filling and emptying indicates that there was a tendency for re-adjustment to take place there as well as in the models. By checking the bolts very carefully and keeping them tight, it was possible to hold this variation well within 10 per cent. In connection with the combined moment of inertia of the stiffener and plate, supplementary tests were run to determine what proportion of the plate would act monolithically with the stiffener. From tests made on flat sheet stiffened with two channels back to back, bolted together and to the plate, it was found that on an average about eight times the thickness of the plate outside of each row of bolts could be considered as acting monolithically with the stiffener so as to give computed load-deflection and

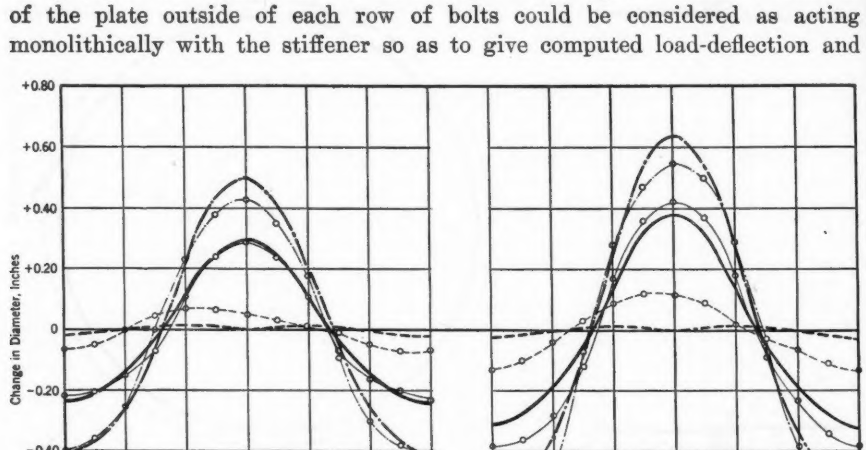

FIG. 31.—CHANGE IN DIAMETER AT SUPPORTS OF MODEL OF SANTEETLAH PIPE LINE.

load-stress values that agreed with experimental results. This value was used in the computations mentioned herein.

A second factor which enters into a comparison between the theoretical and measured deflections or stresses is that the model pipes did not fit the saddles exactly and thereby permitted the pipe to deform until this discrepancy was rectified before the type of action theoretically assumed would begin. The maximum discrepancies were in the order of 0.01 or 0.02 in., which would not seriously affect the distortion of the shell with 66° and 90° saddles; but it would make a decided difference for the 180° saddles since it was assumed that the horizontal diameter of the 180° saddles would not change, which would be the case for the pipe shell if rigidly attached to the saddle or a perfect fit was obtained.

Another factor which entered into the measurements was that the pipe was tested in a room of ordinary temperature, but was filled with water at

a considerably lower temperature. The temperature difference between the water and room was occasionally as great as 15° cent. Such variations made stress determination rather difficult because of the large correction necessary for temperature variation. In spite of these factors the curves given in Fig. 31 show that the agreement between the actual measured and computed distortions is quite close.

Table 1 gives the measured and computed stresses in the stiffeners for two span lengths on the three types of saddles. In Item 4, Columns (4) and (6), a highly localized yielding was noted beside the gauge line. This caused the stiffener to pull away from the pipe shell so as to deflect noticeably past the edge of the support, thereby relieving the stress in the stiffener. The 180° saddles showed a clearance of 0.01 to 0.02 in. at the sides. This would tend to make the stresses at the top and sides of the pipe with the 180° saddles greater than the computed stress.

TABLE 1.—MEASURED AND COMPUTED STRESSES IN A STIFFENED MODEL PIPE LINE 12 INCHES IN DIAMETER

(In kips per square inch.)

Item	Position	Span length, in inches	66° Saddles		90° Saddles		180° Saddles	
			Measured	Computed	Measured	Computed	Measured	Computed
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1......	At top of pipe............	36	8.0	6.2	6.6	4.5	2.6	1.0
2......		108	16.8	12.9	10.1	9.4	......	2.0
3......	Near edge of saddles......	36	16.3	17.1	8.4	12.0	3.2	2.6
4......		108	18.6	36.0	12.0	26.0	......	5.6

A consideration of the bending moments in the shell of the pipe, supported by saddles, reveals the fact that relatively high stresses must always occur at the ends of the saddles. This phenomenon was also quite apparent in the prototype and in the models. The nature of the theoretical distortion curve shows that if the edge of the saddle were free to move and, at the same time, the center portion of the saddle were also relieved, the change in curvature would be greatly reduced. This observation suggested that the pipe line may be supported by struts attached to stiffener rings at some point below the horizontal diameter and in so doing minimize the moments throughout the ring.

In order to eliminate excessive stresses in the top of the strut caused by the twisting resulting from the distortions of the shell, the horizontal component of the resultant reaction of the strut was chosen so that there would be no rotation of the top of the strut under a full water load. Since the strut would probably be joined to the stiffener by a gusset-plate, a concentration of bending moments at the point of support would be eliminated because of the distribution of the forces through the gusset-plate. For the purposes of analysis, however, the resultant reaction of the strut was considered as acting at a point on the neutral axis of the stiffening ring and pipe shell combined. Again applying the theory of elasticity, involving

the consideration that the member is curved, three equations involving the moment at the top, the tangential force at the top, the horizontal component of the strut reaction, and the angular position of the point of application of the strut reaction are obtained.

For various values of the angular position, α, corresponding values for bending moment and tangential force at the top of the pipe and horizontal component of strut reaction were found. By plotting these values against the corresponding values of α it was found that the moment at the top of the shell will pass through zero for some particular α and for the same α the other values would be nearly a minimum. From this it may be concluded that if the resultant reaction of a strut intersects the neutral axis of the combined stiffener and shell at a point 117.3° below the top center, the bending moment at the top will be practically zero and that at the point of support will be nearly a minimum. The reasons involved in choosing this point for the application of the strut reaction may be enumerated, as follows:

1.—Since the point of application of the struts does not rotate (a condition imposed by the choice of the horizontal component of thrust), the slope at this point does not change and, therefore, the distortion must be a maximum.

2.—The distortion of the shell is caused by changes in curvature which, in turn, are caused by bending moments.

3.—The bending moment at the top of the shell is indicative then of the change in curvature of the top and, consequently, to a degree an indication of the distortion.

4.—The magnitude of the distortion at the top is indicative of the magnitude of the distortions throughout the shell.

5.—Consequently, by making M_1 as small as possible, the distortions and stresses are kept small.

Using the value of $\alpha = 117.3°$, equations for the bending moment throughout the ring have been determined. The bending moment at any point in the stiffening ring may be expressed by the following equations:

For values of u between 0 and α,

$$M_u = qR^3L\,[0.825 - 0.825\cos u - 0.500\,u\sin u] \quad \ldots\ldots\ldots (178)$$

and for values of u between α and π,

$$M_u = qR^3L\,[0.564 - 0.452\cos u + 1.571\sin u - 0.500\,u\sin u] \quad \ldots (179)$$

Again using the fundamental differential equation, (174), the equations for the distortion of the shell are found to be:

For values of u between 0 and α,

$$\rho = \frac{q\,R^5\,L}{E\,I}\,[0.825 - 0.809\cos u - 0.537\,u\sin u + 0.125\,u^2\cos u] \quad .(180)$$

and for values of u between α and π,

$$\rho = \frac{q\,R^5\,L}{EI}\,[-\,0.568 + 0.641\cos u + 1.103\sin u - 0.785\,u\cos u - 0.351\,u\sin u + 0.125\,u^2\cos u] \quad \ldots\ldots\ldots\ldots\ldots (181)$$

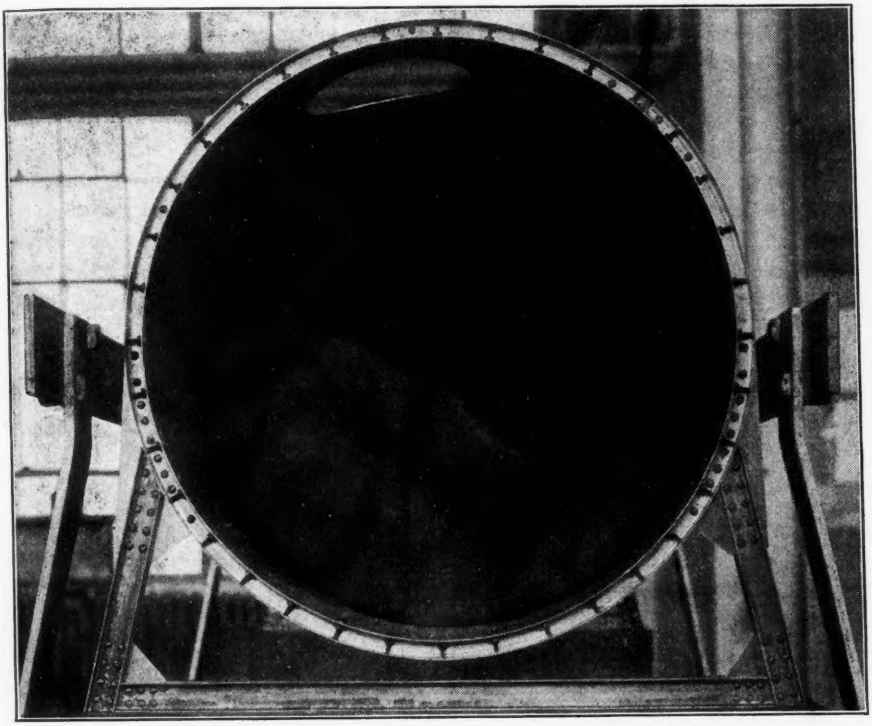

FIG. 32.—END VIEW OF PIPE SUPPORTED BY SPECIAL STRUTS.

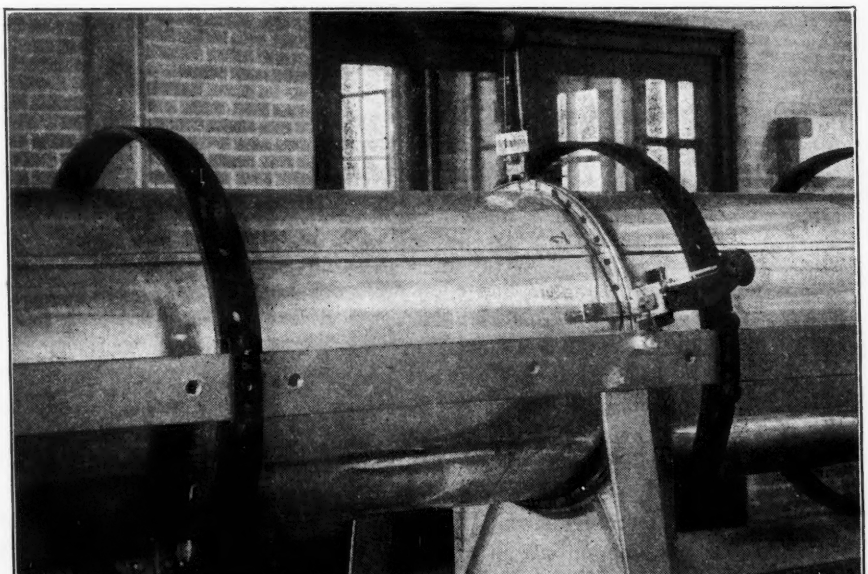

FIG. 33.—ARRANGEMENT OF TENSOMETERS FOR MEASURING STRESSES IN THE STIFFENING RINGS ON A MODEL PIPE LINE.

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Because the distortions of the pipe on this type of support are so small it would be desirable to make corrections in the computed distortions for the temperature change. A very close approximation to the actual distortion caused by temperature change may be expressed by the equation,

$$\rho = \Delta\,(4.29 - 3.37\cos u - 3.37\,u\sin u] \quad \text{.........}(182)$$

in which, Δ equals the change in diameter caused by the temperature change.

In order to check the validity of this theoretical investigation, another model was made using the same scale as for the previous models. The stiffeners were again bolted to the shell and the struts joined to the stiffeners by gusset-plates. Fig. 32 shows an end view of the model built to check the theory. The distortions and stresses of the model were measured again in the same manner as in former tests, with particular consideration given to temperature changes. Table 2 shows the measured and computed stresses

TABLE 2.—MEASURED AND COMPUTED STRESSES IN THE OUTER FLANGE OF THE STIFFENERS OF THE MODEL PIPE LINE ON STRUT SUPPORT WITH 108-INCH SPAN

(In pounds per square inch.)

Angle around pipe from top, in degrees	Measured stresses	COMPUTED STRESSES		Total computed stresses
		From water load only	From temperature change only	
0	— 500*	0	—230	— 230
30	+ 160†	+1 400	—150	+1 250
70	+1 800†	+1 900	+180	+2 080
80	+ 500*	0	+300	+ 300
Outside of strut	— 500*	— 750	+400	— 350
Inside of strut	....	— 750	—400	—1 150

* Average of two or more determinations.
† Single determination.
\+ Denotes tension; — denotes compression.

at different points around the pipe shell. Fig. 33 shows the arrangement of "tensometers" previously mentioned herein, for measuring the stresses in the stiffening rings on a model pipe line.

Fig. 34 shows the computed distortions with three sets of distortions measured on the same ring with the same span. The distortions were so small that the scale necessary to show them is ten times that used for the saddle type of supports. If the same scale had been used it would be impossible to detect the variation in distortion resulting from this type of support because they would all lie so near the zero axis. It was concluded, therefore, that for all practical purposes the distortions are negligible. In view of the agreement between observations on the model and computed values, the writers believe that the theory may be considered a reliable guide in the design of supports and stiffeners for large, thin-shelled cylindrical containers.

From the nature of the moment equation it may be seen that the stresses in the stiffening ring and struts are directly proportional to the span length for any given diameter and that they vary as the cube of the radius of the pipe for spans of equal length. The critical bending moment for design

purposes will be that at the bottom of the shell because the moment at the point of support is not only absorbed by the gusset-plate, but also diminished by the distribution of the strut reaction by the gusset-plate. The maximum bending moment at the bottom of the shell reduces to,

$$M_{\pi} = 0.112\ qR^3L \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(183)$$

This equation is on the side of safety because of the stiffening effect of the gusset-plates and the fact that the strut reaction is distributed over a con-

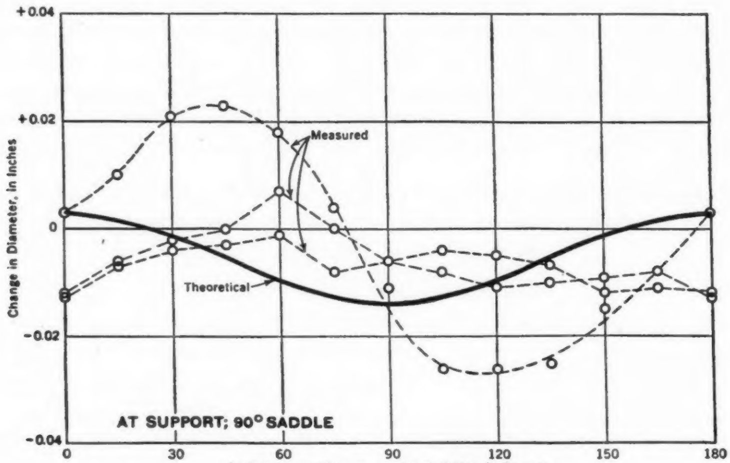

FIG. 34.—MEASURED AND COMPUTED DISTORTIONS OF A MODEL SUPPORTED ON STRUT SUPPORTS SHOWN AS CHANGES IN DIAMETER.

siderable portion of the ring by the gusset-plate. The maximum shear stresses in the side of the shell may be found from the equation,

$$S_{ys} = \frac{qR\ x'\ \sin u}{t} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(184)$$

which reduces to,

$$S = \frac{q\,R\,L}{2\,t} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(185)$$

for the maximum shear stress in the side of the shell.

F. KNAPP,[45] ESQ. (by letter).[45a]—The subject of this interesting paper has been treated[46] previously by Professor Dr. Dieter Thoma, of Munich, Germany, in a paper entitled "Die Beanspruchung freitragender gefüllter Rohre durch das Gewicht der Flüssigkeit." Using the fundamental equations of the theory of elasticity as a basis, Professor Thoma considers the case of a completely filled pipe, either "encastrée" at the ends, or having the end sections connected

[45] Asst. Hydr. Engr., The Saõ Paulo Tramway, Light & Power Co., Ltd., Saõ Paulo, Brazil.

[45a] Received by the Secretary January 9, 1932.

[46] *Zeitschrift* für das gesamte Turbinenwesen, 1920, p. 49.

to rigid rings which may move normally and in the direction of, but not inclined to, the axis. However, he does not consider the design of these rings.

Through the special courtesy of Professor Dr. L. Föppl, of the Technische Hochschule of Munich, the writer was fortunate enough to study, in the original, the work of Dr. R. Abdank, entitled "Die Berechnung freiaufliegender Rohre bei beliebiger Füllhöhe," regarding the stresses in large pipe lines caused by partial filling of the pipe.[47]

Based on the extensive experiments made on a model pipe for the Shannon Development, in Ireland, Dr. Abdank studies the following two cases:

(1) That in which the inclination of the pipe is such that partial filling is possible without the water level cutting the supporting rings. (In this case, Professor Thoma's formulas for a completely filled pipe may be applied with safety); and,

(2) That in which the pipe is horizontal. In Case (2) the maximum normal stress in the membrane in the direction of the axis of the pipe and for a freely supported pipe, may be calculated by the approximate formula:

$$f_{(max.)} = \frac{3}{32} L \times \frac{L}{t} \sqrt{\frac{R}{t}} \quad \dots\dots\dots\dots\dots\dots (186)$$

In the case of a continuous pipe, this stress is reduced to $\frac{2}{3}$ $f_{(max.)}$, tension at the connection of the ring, and to $\frac{1}{3}$ $f_{(max.)}$, compression midway between the rings. These stresses are always greater than in the case of a completely filled pipe.

[47] Submitted as a thesis in partial fulfillment of requirement for the degree of Doctor of Engineering at the Technische Hochschule, of München. An abstract of this thesis was published by Dr. Abdank in *Die Bautechnik*, for June 19, 1931.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

SOIL MECHANICS RESEARCH[1]

Discussion

By Messrs. D. P. Krynine, J. Stuart Crandall, and Frank Alwyn Marston

D. P. Krynine,[42] M. Am. Soc. C. E. (by letter).[42a]—Prior to 1925, when Charles Terzaghi, M. Am. Soc. C. E., published his findings in the field of soil mechanics[43] that science was a rather weak sub-division of engineering mechanics. It dealt with imaginary dry substances affected by friction, and sometimes cohesion. The development of the science was accelerated when it was emphasized that the backbone of engineering soil research is the study of the physical properties of the material.

Rôle of Soil Physics.—In the present epoch a new science of soil mechanics is being created and no necessary tool should be disregarded. Soil physics has proved to be a mighty "base" on which engineering theories may be erected; therefore, a thorough knowledge of it should not be underestimated. Obviously, such knowledge is economically worthless, as Professor Gilboy rightly states, unless it provides a means for answering engineering questions. Unfortunately, the branch of soil physics that may happen to be necessary in the future is not known at present; and a modern worker in soil engineering research should be able to study any new branch of soil physics, at any time, and even to develop it independently if necessary. For this purpose a certain training and familiarity with soil physics and related sciences are required.

Some time ago soil colloids were considered to be the cause of many phenomena in soils. Professor Terzaghi did not hesitate to plunge into a detailed study of the mechanics of absorption, and he published a paper on the subject that would have done credit to any specialist in physical

Note.—The paper by Glennon Gilboy, Jun. Am. Soc. C. E., was presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929, and published in October, 1931, *Proceeding*. Discussion of the paper has appeared in *Proceedings*, as follows: December, 1931, by Messrs. J. C. Meem and H. de B. Parsons; January, 1932, by Messrs. Jacob Feld and John R. Jahn; and February, 1932, by Messrs. C. H. Eiffert, A. A. Eremin, F. N. Menefee, and E. G. Walker.

[42] Research Associate in Soil Mechanics, Dept. of Civ. Eng., Yale Univ., New Haven, Conn.

[42a] Received by the Secretary December 4, 1931.

[43] "Erdbaumechanik auf bodenphysikalischer Grundlage," by Charles Terzaghi, Leipzig and Vienna, 1925.

chemistry.[44] The attitude of any modern soil engineer should be similar; otherwise, the development of the science will be retarded. However, his writings, addressed to the general technical public, should be different from those addressed to specialists. They should be concise and should contain as little as possible of complicated formulas and physical speculations not directly applicable in engineering practice.

Mechanical Analysis.—Civil engineers received their first knowledge of soil research procedure from the agriculturalists, and from that source came a tendency to over-estimate the value of mechanical analysis. Those who work in soil mechanics know how much time has been spent in studying and improving methods of making mechanical analyses.

Considerable research on sedimentation has been done by most investigators. Sedimentation methods depend on the fact that in a viscous medium, such as water, particles sink at different speeds depending on their weight, specific gravity, and configuration. The relation between their diameter and their velocity of sinking is given by the well-known formula introduced by Stokes.[45] This equation is valid for spherical particles. However, clay particles are scale-like and, therefore, Stokes' formula can give only the "effective diameter" of the particle, which is defined as the diameter of an imaginary sphere of the same material which would sink in water with the same velocity as the given particle. The general idea of the test consists in stirring the soil solution well and allowing it to stand for a given time, t, when the turbid liquid is poured or siphoned into a receptacle; it does not contain particles coarser than a certain diameter, d, computed from Stokes' formula.

The quantity of suspended particles in the turbid liquid may also be measured directly, without pouring or siphoning (hydrometer method). Results obtained by even the most refined sedimentation methods, however, are only approximate. The Russian physicist, G. I. Pokrowski, studied[46] some optical properties of soils and found that variations of such properties correspond only to variations in the size of particles more than 0.3 microns in magnitude. In other words, it seems to be inadequate to apply Stokes' formula to particles less than 0.3 microns in magnitude, that is, to small collodial particles and to those somewhat coarser. The walls of the container exert a certain influence on the particles according to their electric charge; and the path of sinking particles is not at all straight, as Stokes' formula assumes.

The conclusion to which engineering soil investigators have come lately, may be formulated as follows. Since complicated methods give only approximate results, it is preferable to save time and energy by using simpler methods provided the degree of accuracy remains practically the same. In this way, the methods of Wiegner, of Gessner, of Sabanine (used in Russia),[47]

[44] "The Mechanics of Adsorption and of Swelling of Gels," by Charles Terzaghi, Colloid Symposium Monograph, 1926.

[45] "The Physical Properties of Soil," by Bernard A. Keen, Lond., 1931, p. 39 *et seq.*

[46] "Optical Investigation of Soil Suspensions," by G. J. Pokrowski, Soil Investigations in 1927 (in Russian), Moscow, 1928, p. 57 *et seq.*

[47] "Textbook on Highway Engineering," by D. P. Krynine, Third Edition (in Russian), Moscow, 1930, p. 190.

of Robinson, and others, which may be adequate for agricultural purposes, have become merely of historical interest to a civil engineer. For engineering routine research, sieve analysis for coarser particles and the Bouyoucos hydrometer test for minute particles are to be recommended.

A full description of methods of procedure for mechanical analysis for the combined sieve and hydrometer method has been published by the U. S. Bureau of Public Roads.[48] In applying the Bouyoucos test, proper use of deflocculating agents should be emphasized (sodium silicate solution for soils having a plasticity index of less than 20 and hydrogen peroxide for heavier clays). Professor Bouyoucos himself describes[49] this procedure in a manner somewhat different from that of the Bureau of Public Roads.

The accumulated percentages of particles of different effective diameters may be plotted to a logarithmic scale as shown in Fig. 1 of Professor Gilboy's paper, and thus a particle—size accumulation curve—may be obtained. In his routine research for highway purposes the writer seldom makes the complete hydrometer test. It has been felt that in the majority of practical cases, the 15-min. Bouyoucos test is all that is needed. Admittedly this is rather empirical; but it gives a fair idea of the amount of colloidal matter in the soil. This is especially true of low percentages when obtained in the 15-min. test.

Moisture Equivalent.— The writer understands that no research on moisture equivalent has been made at the Massachusetts Institute of Technology. Possibly this fact may be explained by the lack of interest in this item on the part of foundation engineers. On the contrary, highway engineers consider the study of the moisture equivalent as an important feature. For reasons which follow, the writer wishes to suggest the importance of a certain familiarity with the moisture equivalent in foundation engineering as well.

The moisture equivalent represents the percentage of water, based on the dry weight of the soil sample (usually 5 grammes), retained by the soil after first being soaked in water for about 6 hours, then drained in a humidifier for 12 hours, and, finally, subjected to the action of a centrifugal force with an acceleration one thousand times that of gravity, for 1 hour. This is the procedure of the U. S. Bureau of Public Roads; the duration of soaking and draining, as recommended by the U. S. Bureau of Soils, is different. The centrifuge method was introduced by Messrs. Lyman J. Briggs and John McLane[50] about 1907; a few years ago a field method was proposed[51] by A. C. Rose, Assoc. M. Am. Soc. C. E. However, on applying the centrifuge method difficulties arise. In the case of heavy clays, especially of

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[48] "Procedures for Testing Soils, etc.," by A. M. Wintermeyer, E. A. Willis, and R. C. Thoreen, *Public Roads*, Vol. 12, No. 8 (1931), p. 198 *et seq.*

[49] "A Comparison of the Hydrometer Method and the Pipette Method, etc.," by George John Bouyoucos, *Journal*, Am. Soc. of Agronomy, Vol. 22, No. 8 (1930), p. 747 *et seq.*

[50] "The Moisture Equivalent of Soils," by Lyman J. Briggs and John McLane. U. S. Dept. of Agriculture, *Bulletin 45*, Washington, D. C., 1907. A great number of papers along these lines are to be found in *Soil Science* and other magazines.

[51] "Foundations and Drainage of Highways," by Albert C. Rose, *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 64 *et seq.*

the sticky type, the water is not thrown out through the holes in the bottom of the container (Gooch crucible), but collects on the top of the layer. This phenomenon, called "water-logging," should be of interest to foundation engineers. According to computations by Professor Terzaghi the process of centrifuging is equivalent to the application of a load equal to 2 kg. per sq. cm.

The theory of hydrodynamic phenomena in soils requires the pressing out of water under load. Possibly in a loaded soil layer the tendency is for water to be squeezed out at both top and bottom. Owing to the sticky nature of some gels the clay may become impermeable at the bottom of the layer, or close to it. This is analogous to the phenomenon of water-logging in the process of centrifuging. The same may happen close to the top, or in the middle, of a layer. Thus, instead of being thrown out, water remains in the interior of the layer, which may conserve its voids-ratio close to the liquid limit. Such cases have been disclosed in some borings and, as far as the writer knows, have not been thoroughly understood.

Further study will possibly establish closer relationship between the values of both the moisture equivalent and the coefficient of permeability. Furthermore, there is interdependence between the contrifuge moisture equivalent and the colloidal content of the soil as determined by the 15-min. Bouyoucos test. This fact has been stated by Professor Bouyoucos;[52] an analogous interdependence has been found by the writer in testing more than one hundred samples of soils in New Haven County, Connecticut.[53]

Limits of Consistency.—The procedure of determining the limits of consistency has been fully described in *Public Roads.*[48] The method of determining the liquid limit is stated as follows: "The dish is held firmly in one hand, with the groove parallel to the line of sight, and tapped lightly with a horizontal motion against the palm of the other hand ten times." Obviously, the apparatus constructed by Mr. A. Casagrande[54] minimizes the personal equation and is therefore superior to that adopted by the Bureau of Public Roads. The Bodman and Tamaschi method[55] involves dropping a flat-bottomed, straight-sided, metal container (50 mm. in diameter, and 13 mm. high), holding the plastic mass of soil (weight, 20 grammes when in water-free state), from a height of 30 cm. A ditch of triangular cross-section is excavated along the dish with two razor blades held at an angle of 45° to the earth's surface. The number of impacts is plotted against the moisture content. The equation of the curve is:

$$N = aw^n \qquad (7)$$

[52] "A New, Simple, and Rapid Method for Determining the Moisture Equivalent," by George John Bouyoucos, *Soil Science*, Vol. 27 (1929), p. 234 *et seq.*

[53] "Some Principles of Soil Surveying and Soil Mapping for Road Purposes," by D. P. Krynine, Paper presented to the Highway Research Board, National Research Council, in December, 1931.

[54] "Soil Mechanics Research," by Glennon Gilboy, *Proceedings*, Am. Soc. C. E., October, 1931, p. 1169.

[55] "Studies of Soils in the Plastic State," by G. B. Bodman and M. Tamaschi, *Soil Science*, Vol. 30 (1930), p. 175 *et seq.* Another example is the method of Mr. Roy C. Roberts, as described in the Report of the Am. Soil Survey Assoc., Ames, Iowa, 1930, p. 56 (mimeographed copy).

in which, N is the number of impacts needed to fill the ditch; w, the percentage of water as referred to the dry weight of the sample; and a and n are constants. Obviously, the Bodman and Tamaschi method has much in common with the idea developed by Mr. Casagrande.

There have been many other attempts to measure plasticity, advantage of which may be taken by a soil engineer. Dr. J. W. Mellor takes[56] a clay ball of standard size, applies a sufficient stress on a standard area to cause steady compression, and measures the total motion required to split the ball open at the edges. Thus, two factors may be obtained—stress and strain—both depending on the water content. Plasticity is expressed by the product of these two factors, when this product reaches a maximum. In Bischoff's test,[57] mixtures of soil powder, sand, and water are prepared. After drying, a moist brush is passed twenty-five times back and forth on the surface of each cylinder. Some of the cylinders (the bulging ones) will still remain convex, while others will become concave. Suppose that after the test there are n convex cylinders; then, according to Bischoff, n is the plasticity coefficient. In the method introduced by Messrs. J. W. Talwalkar and C. W. Parmelee[58] a plastic clay bar is subjected to torsional stresses, and a curve showing the stress-strain relations is drawn. There are still other methods, as, for instance, that of Professor Zemiatchensky.

Attempts have been made to explain soil plasticity by colloidal content. Mr. M. L. Nichols[59] has found experimentally that the lower plastic limit and the plasticity index depend on the colloid content of the soil. According to Mr. Nichols, the lower plastic limit decreases with the increase of the "activity" of the colloids acting in a given soil; and the plasticity index (or "plasticity number") may be expressed by an exponential function of the colloid content. According to Mr. Nichols' computations, a soil with 15 to 17% colloid content would have a plasticity index of 1; and this is quite closely in agreement with practical data. The writer does not mean that the attempt to explain so many soil properties by its colloidal behavior should be revived in soil mechanics; but he wishes to point out that colloid content determined by even a rudimentary method represents a valuable datum which should never be lost sight of by the engineering soil investigator.

Influence of Scale-Like Particles.—Atterberg has found[60] that only soils having disk or plate-shaped (scale-like) particles showed signs of plasticity. Mr. L. D. Baver has attributed[61] plasticity to the orientation of plate-shaped colloidal particles. The influence of scale-like particles, such as mica, on the compressibility of soils has been evidenced by the research of Professor

[56] "On the Plasticity of Clays," by J. W. Mellor, *Transactions*, Ceramic Soc. (British), Vol. 21 (1921–22), p. 93–94.

[57] "Die feuerfeften Tone," by Bischoff, 1904, p. 84.

[58] "Measurement of Plasticity," by T. W. Talwalkar and C. W. Parmelee, *Journal*, Am. Ceramic Soc., Vol. 10 (1927), p. 670.

[59] "An Explanation of Dynamical Properties of Soils," by M. L. Nichols, *Agricultural Engineering*, Vol. 12, No. 7 (1931), p. 259 *et seq.*

[60] "Die Konsistenz und die Bindigkeit der Böden," by A. Atterberg, *International Mitteilungen Bodenkunde*, Vol. 2 (1912), p. 149 *et seq.*

[61] "The Atterberg Consistency Constants," by L. D. Baver, *Journal*, Am. Soc. of Agronomy, Vol. 22 (1930), pp. 935–948.

Terzaghi and Professor Gilboy.[62] Scale-like colloids belong to the same category as mica. Possibly, Baver's opinion[61] relative to plasticity should be generalized and applied to compressibility as well. In other words, compressibility of soils should be explained, not by the "scale-likeness" of particles as such, but by their orientation, or by the possibility of such orientation in applying the load.

In order to clarify the idea of orientation, suppose a small paper slip is dropped on the floor. Its path of falling is more or less vertical, and the slip will be deposited with its face touching the floor; that is, it will be oriented horizontally. Let a great number of small paper slips be dropped simultaneously. Although most of them will be oriented on the floor, some will not be so oriented. Such is the case of sedimentary clays consisting of small scale-like particles; and the degree of orientation can be different in different clays. Therefore, in a disturbed sample all the particles are mixed and the degree of orientation has little to do with the natural state. Obviously, in using disturbed samples in the form of powder, not only the degree or orientation, but the voids-ratio and the character of the particle itself are not the same as in Nature. It has already been stated that according to Stokes' formula clay particles lying higher should be finer, as a rule, than those from lower layers. Therefore, if particles possessing different properties are mixed so as to form a "disturbed" sample, the testing would scarcely give very desirable data. The initiative of Professor Gilboy in studying the question of how to obtain large undisturbed samples is welcomed.

An attempt of Dr. G. J. Pokrowski to express the degree of orientation numerically should be mentioned.[63] The intensity of polarized light reflected by the surface of the soil was proved to depend on the degree of orientation. The latter may be measured by the ratio of two light intensities reflected in predetermined directions. The maximum value of this ratio was proved to be equal to 1.13 (talc), and it varies between 1.03 and 1.07 for clays.

Permeability.—Darcy's law (Equation (2) of Professor Gilboy's paper) may be written in the form:

$$v = \frac{Q}{At} = k\,N \qquad (8)$$

in which, v is the velocity of flow. Forchheimer showed[64] that generally:

$$v^m = k\,N \qquad (9)$$

in which, the limits of m are 1 and 2.

Smreker[65] proposed an equation in the form of Equation (9):

$$N = a\,v^2 + b\,v^{\frac{3}{2}} \qquad (10)$$

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[62] "The Compressibility of Sand Mica Mixtures," by Glennon Gilboy, *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 555 *et seq.*

[63] On Orientation of Particles in Soils," by G. J. Pokrowski, Soil Investigations in 1928–1929 (in Russian), Moscow, 1930, p. 88.

[64] "Wasserbewegung durch Böden," by Ph. Forchheimer, *Zeitschrift* des Vereins Deutscher Ingenieur, Vol. 45 (1901), p. 1781 *et seq.* This formula is also mentioned by Forchheimer in "Hydraulik," Berlin, 1914, p. 420.

[65] "Das Grundwasser, seine Erscheinungsformen, Bewegungsgesetze, und Mengenbestimmung," by O. Smreker, Leipzig und Berlin, 1914, pp. 29–33.

in which, a and b are constants. Afterward, he omitted the first member in the right-hand side of Equation (10) and obtained:

$$N = b\ v^{\frac{3}{2}} \qquad \text{(11)}$$

This formula provoked considerable discussion in Europe.

On proposing his formula Darcy had a limited application[66] in mind; therefore, its application in other cases may serve only as an approximation. Actual research, however, is generally based on the assumption that Darcy's law is an exact one. In this connection, it should also be emphasized that the coefficient of permeability, k, is not a constant, but fluctuates during the percolation. In the beginning of the percolation, the initial value of k decreases. This fact has been established by many investigators, beginning with Forchheimer[64] and others before 1901, to the modern research workers of the U. S. Bureau of Chemistry and Soils in 1931.[67] After a certain time, the value of k may increase again considerably. Strictly speaking, such fluctuations should be taken into account in designing permeameters.

Another inconvenience of the apparatus in Fig. 4 is the possibility of "colloid erosion" in using high values for hydraulic gradients. Under the action of a water column, colloids should migrate downward, rapidly, if the column is high, and slowly, otherwise. A proof of this statement is furnished by Nature herself; as known, colloidal matter from the upper soil horizons has been partly removed to form lower alluvial horizons that are richer in colloids.

The following simple device for determining the coefficient of permeability has been used by the writer for loams and sandy soils in Connecticut. In Fig. 23 there are two metallic containers, A and B. The bottom of the inner cylinder, A, is perforated and covered with a filter paper. The soil sample is placed on this paper so that its upper surface corresponds to the edge of the outer cylinder. The edge serves as an overflow device. First, the outer cylinder is filled full of water, and the sample is allowed to absorb as much as it can. Additional water should be added gradually in order to maintain a constant level in the outer cylinder. Afterward the inner cylinder is carefully filled without disturbing the surface of the sample; percolation begins immediately. Suppose that at a certain time, t, from the beginning of percolation, the water level in the inner cylinder has dropped to a distance, a, from the top. Let, H = the vertical distance from the top of the inner cylinder to the top of the outer cylinder; d = the thickness of the sample; and k = the coefficient of permeability. Then, applying the Smreker formula:

$$k = \frac{3\,d^{\frac{3}{2}}}{t}\left(\sqrt[3]{H} - \sqrt[3]{H - a}\right) \qquad \text{(12)}$$

[66] "Les fontaines publiques de la ville de Dijon," by H. Darcy, Paris, 1856, p. 590.

[67] "A Laboratory Study of the Field Percolation Rates of Soils," by C. S. Slater and H. C. Byers, *Technical Bulletin No. 232*, U. S. Dept. of Agriculture, January, 1931, p. 9.

The height, H, and the thickness of the sample, d, are supposed to be known. For a given value of a,

$$k = \frac{c}{t} \qquad \text{.........................(13)}$$

in which, c is a constant, corresponding to the given value of a.

The apparatus is unusually easy to handle and adequate for rough work. The inner cylinder may be taken out and driven into the ground; and thus an undisturbed sample may be obtained and tested. In order to avoid mistakes in the beginning of the percolation, the writer generally observes the time, t, necessary for the water to drop from a level, $a = a_1$, to another level, $a = a_2$. Both levels are marked with white lines on the inner surface of the

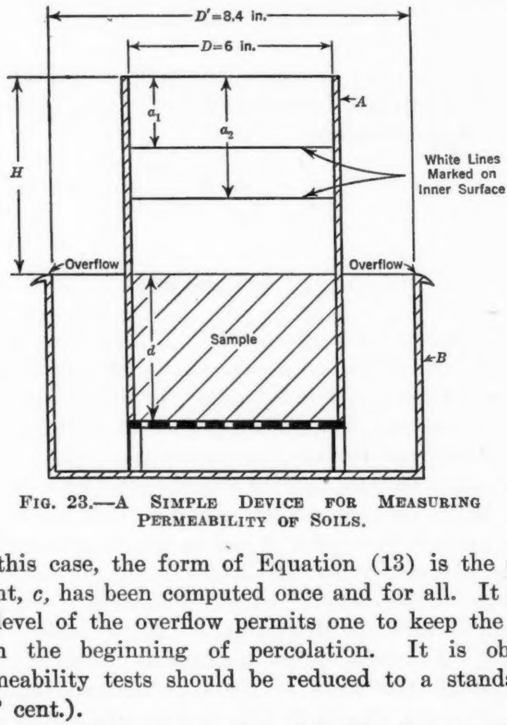

FIG. 23.—A SIMPLE DEVICE FOR MEASURING PERMEABILITY OF SOILS.

cylinder. In this case, the form of Equation (13) is the same as before; and the constant, c, has been computed once and for all. It should be added that the high level of the overflow permits one to keep the sample entirely saturated from the beginning of percolation. It is obvious that the results of permeability tests should be reduced to a standard temperature (usually + 10° cent.).

The horizontal capillarity test proposed by Mr. Casagrande should receive special attention because of its unusual interest (see Fig. 5). From Equation (5) of the paper, the following may be written.

$$\frac{x^2}{t} = C_1 \sqrt{k} \text{ (a constant)} \qquad \text{..................(14)}$$

in which, C_1 (for a given soil in a given condition) is a constant. The value of k is assumed to be constant.

As far as the writer knows, formulas for the speed of capillary movement, both vertical and horizontal, were first developed[68] by Diro Kitao, in 1897.

[68] "Handbuch der Bodenkunde," Edited by E. Blanck, Vol. 6, p. 104 *et seq.;* also, p. 172.

He was also the first to draw the attention of investigators to the meaning of the ratio, $\frac{x^2}{t}$.

Later investigators, for instance, J. S. Kozeny,[69] also paid due attention to the study of the ratio, $\frac{x^2}{t}$. However, the idea of applying observations on capillary movement to the determination of the coefficient of permeability, k, was clearly expressed only in 1925, by Professor Terzaghi.[43] In 1930, Zunker[68] proposed the determination of the value of k from data referring to the downward capillary movement. His formula is:

$$\frac{x^2}{t} = 2k\ (H - Z + L) \qquad (15)$$

in which, Z and L are values of excess atmospheric pressure at the bottom and the top of the capillary column, respectively, and H, the pulling force of the meniscus. Approximately:

$$\frac{x^2}{t} = 2k\ H \qquad (16)$$

or,

$$\frac{x^2}{t} = C_2\, k \text{ (a constant)} \qquad (17)$$

in which, C_2 is a constant for a given soil. The form of the right side of both Equations (14) and (17) being somewhat different, it would be interesting to see a development of the formula. It should also be emphasized, that the capillary saturation, mentioned by Professor Gilboy in introducing Fig. 5, does not accompany the moving meniscus at all. Saturation takes place gradually from the entrance of the experimental tube to its outlet; and sometimes many hours are needed for complete saturation. Observations were made on the gradual saturation of soil in the experimental tube without extracting samples therefrom for the determination of moisture content.[70] For this purpose the "reflectometer" of Dr. Pokrowski[71] was used. This is an apparatus which permits the determination of the moisture content of a given soil by observing certain optical phenomena on the visible surface of the moist soil through the walls of the experimental tube. Progressive saturation curves for tubes, 17 and 23 mm. in diameter, are shown in Fig. 24. In these curves, h is the time, in hours, from the beginning of capillary movement. The visible moisture line is not shown.

Compressibility and Consolidation.—The theories of soil mechanics referring to compressibility and consolidation, and the consolidation apparatus shown in Fig. 7, have been developed almost exclusively by Professor Terzaghi.[43] Fig. 25 shows, diagrammatically, the general installation of the

[69] "Ueber den kapillaren Aufstieg des Wassers in Boden," by J. S. Kozeny, Sitzungsberichte Akademie Wissenschaft, Wien, Vol. 136 (1927), p. 271 *et seq.*

[70] "Penetration of Water into Clay," by D. P. Krynine, Soil Investigations in 1928–1929 (in Russian), Moscow, 1930, p. 41 *et seq.*

[71] "The Reflectometer," by G. J. Pokrowski, Soil Investigations in 1928–1929 (in Russian), Moscow, 1930, p. 80 *et seq.*

same apparatus at Yale University: *A* is a wooden plank fixed by the clamps, *B*, to a solid laboratory table, *T*. A metallic column, *C*, carries the axis of rotation of a lever, *L*, on the other side of which a variable load, *P*, may be applied (bags containing different quantities of shot). Thus, loads may be applied to the experimental cylinder, *E*, in definite increments. The installation is very simple and inexpensive.

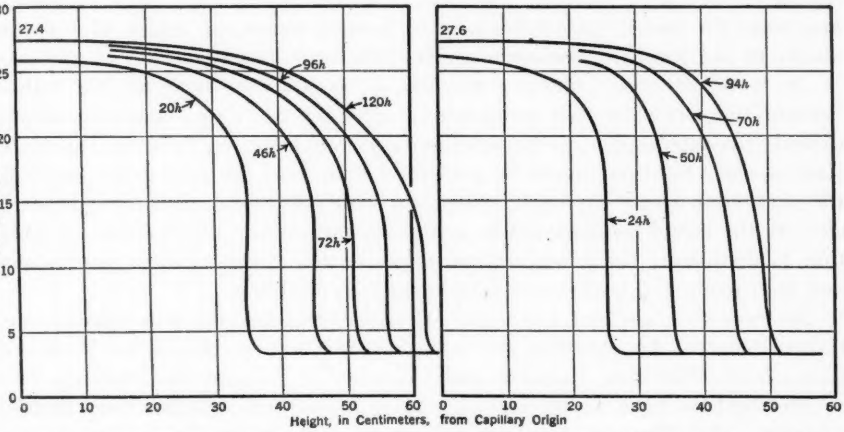

FIG. 24.—PROGRESSIVE CURVES OF CAPILLARY SATURATION IN A GLACIAL LOAMY CLAY.

In the writer's opinion the shape of the curves obtained for samples $\frac{1}{2}$ in. thick, as shown in Fig. 6, may be considered only an indication (not conclusive) of the consolidation of a natural sedimentary clay layer. Obviously, it is understood that the difference in thickness should be taken into account.[45] A simple consideration of Stokes' formula leads one to the conclusion that, as a rule, a sedimentary clay layer cannot be homogeneous. The water in the basin where such a clay is being formed, is practically a col-

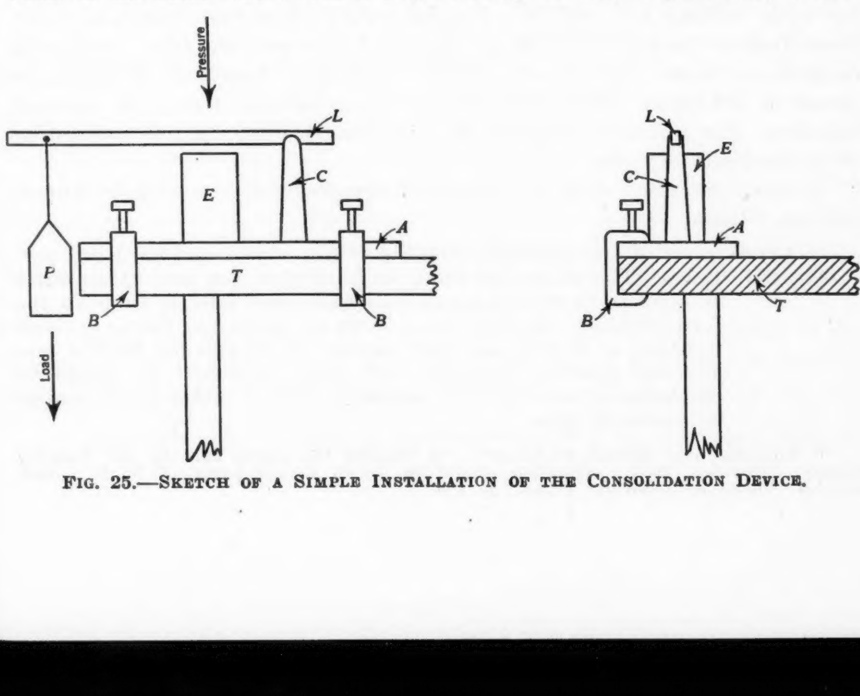

FIG. 25.—SKETCH OF A SIMPLE INSTALLATION OF THE CONSOLIDATION DEVICE.

loidal suspension; colloids settle gradually to the bottom, flocculating or simply mixing with fresh material reaching the basin from eroded localities. The influx of the eroded material is of intermittent intensity, and possibly it stops periodically. The topography of the bottom of the basin and the water level of the basin may change. In this connection superficial crusts may sometimes be formed. The mechanics of the formation of such crusts was developed by Professor Terzaghi.[43] Furthermore, clay may be sticky, and traps for percolating water may be formed under the action of loads as stated in the foregoing remarks under "Moisture Equivalent."

As a result, some faulty permeable or completely impermeable colloid "sheets" may exist in the interior of a clay layer. This sometimes should disturb the whole picture of consolidation. The writer believes, therefore, that special attention should be paid to the study of the geological profile in the particular case. Especially, places at which the natural water content is close to the liquid limit should be examined carefully. These places in addition to disturbing the consolidation, may reveal the existence of lateral flow and thus cause the unexpected settlement of a building.

Internal Friction and Cohesion.—Both internal friction and cohesion are responsible for the shearing resistance of the soil as stated by Professor Gilboy (see, "Internal Friction and Cohesion"). Numerous investigations of friction have been made; and its nature is somewhat clearer than that of cohesion. An adequate definition of cohesion was given by Newton in his "Opticks," before 1721; and about 1773 the laws of friction and cohesion, as affecting a mass of soil, were formulated by Coulomb.[37] To-day, cohesion is being studied to a considerable extent by metallographists and physicists working in that field. Although some progress has been made, the question of cohesion in metals seems still to be open. Further details may be found in the publications of the Faraday Society.[72] The problem of cohesion appears to be even more complicated in soil mechanics than in metallography. In this case the investigator has to deal not with a "continuum" as in metals, but with millions and millions of small bodies, often surrounded by water films stuck to the surface of the particle, and separated one from another by cushions of water and air of variable thickness. Professor Terzaghi, as stated by Professor Gilboy, has distinguished between true and apparent cohesion. For practical purposes the apparent cohesion only is interesting in the majority of cases.

A short statement as to the nature of apparent cohesion may be formulated as follows:

(*a*) A mass of soil generally consists of air, water, and hard soil particles. The molecular forces of hard particles may cause water to stick to their surfaces; or particles may adsorb water in the form of films. In their turn, there are molecular forces between particles of water; and these forces are responsible for the tensile and shearing resistance of water in films. It should be emphasized that water in capillary films is rather hard, similar to hardened glue.

[72] "Cohesion and Related Problems": A General Discussion held by the Faraday Society, November, 1927. Attention should be drawn to the paper of N. K. Adam, entitled "Cohesion in Surface Films," p. 149.

(*b*) The molecular forces act within a very short distance from the surface of a particle (range of molecular forces). They are strong close to the surface and rapidly decrease toward the boundary of their range. When a surface of separation (or surface of rupture) passes between two particles surrounded by water films, a phenomenon takes place analogous, to a certain extent, to the shearing of a beam, relatively weak in the middle of the span and very strong on the ends (Fig. 26). In cohesive soils this span is microscopically small. The smaller the span (that is, the distance between the particles), the stronger is the beam, principally because of the dropping of the weak part from the middle.

The following conclusions may be drawn from Fig. 26, which may easily be checked in practice: (1) Ramming or packing of soil increases its cohesion, because of the decrease of the span of the imaginary beam (Fig. 26); (2) settlement of soil produces the same effect, for the same reason; and (3) saturation of soil with water makes the span of the imaginary beam increase;

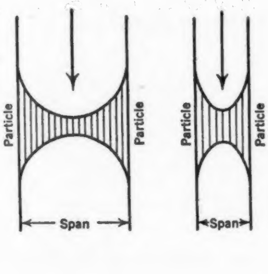

FIG. 26. — GRAPHIC REPRESENTATION OF INCREASE OF SHEARING RESISTANCE WITH A DECREASE OF DISTANCE BETWEEN PARTICLES.

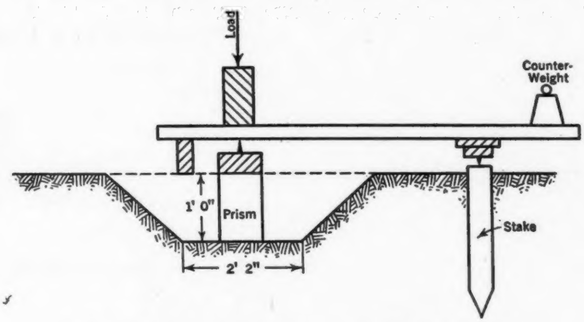

FIG. 27.—DETERMINATION OF SHEARING RESISTANCE OF THE SOIL AFTER BUISMAN.

as a result, cohesion is reduced. Thus, the principal cause of cohesion in soils is water in proper proportion, capillary or even hygroscopic. There is also an opinion that a mineral binder may contribute to the increase of cohesion of smaller particles.[73]

Although Professor Gilboy does not formulate conclusions drawn from working with the machine of Figs. 10 and 11, the few results he cites are very interesting, and that referring to the angle of friction of clays, should be especially emphasized. Contrary to a widespread opinion, the angles of friction of clay, ϕ, appear to be considerable. Perhaps this is the eve of important events in this branch of research.

The existing methods of determining cohesion are principally those of the laboratory. The well-known idea of pulling one box of soil along the surface of another was developed by the late William Cain, M. Am. Soc. C. E., and

[73] "Interrelationship of Load, Road, and Subgrade," by C. A. Hogentogler, Assoc. M. Am. Soc. C. E., and Charles Terzaghi, M. Am. Soc. C. E., *Public Roads*, Vol. 10, No. 3 (1929), p. 37 *et seq.*

has appeared many times in literature. The method based on the rupture of a cylinder (Fig. 9), was elaborated by Professor Janicsek, of the Technical University of Hungary, when working under the supervision of Professor Terzaghi, at Cambridge, Mass. The procedure in question is simple and efficient, but inconvenient in some respects. In addition to a certain difficulty in measuring the angle, α, accurately, there remains unconsidered the influence of the friction between the top and bottom of the experimental cylinder, and the compression machine, as Professor Janicsek himself points out in his writings.[74]

Professor A. S. Buisman, of Holland, finds the coefficients of friction and cohesion by a similar procedure.[75] In addition to his laboratory work, Professor Buisman made experiments in the field destroying soil prisms as shown

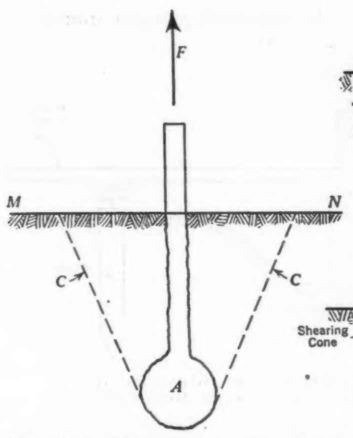

FIG. 28.—POSSIBILITY OF DETERMINING SHEARING RESISTANCE OF SOIL BY PULLING OUT A MALONE ANCHOR.

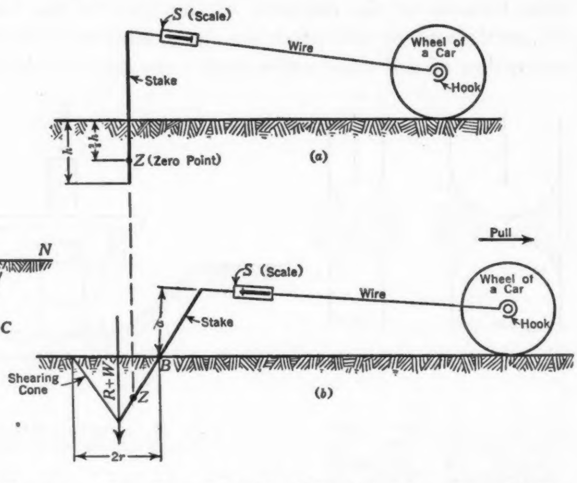

FIG. 29. — SIMPLE METHOD OF DETERMINING SHEARING RESISTANCE OF SOIL BY PULLING A STAKE WITH AN AUTOMOBILE.

in Fig. 27. It is noteworthy that for a clay and a loam, tested in the laboratory, Professor Buisman found the values of ϕ to be between $27\frac{1}{2}$ and 29 degrees.

Among his other experiments, the strength of earth in shear has been studied[76] by John H. Griffith, M. Am. Soc. C. E. He used a method somewhat similar to that proposed by Professor Cain, but dealt with blocks of natural earth about 12 in. wide and 24 in. long. The ratio of stress to strain, or the shearing modulus, was computed, and corresponding curves were drawn.

In 1923, dynamometer tests on a special type of anchors for transmission towers were conducted for the Blaw-Knox Company in Pittsburgh, Pa., and,

[74] "Alkalmas—e a Koska a Töröszilárdság Megallapitásáparà?" by Jozsef Janicsek, *Technika*, Vol. 8, Nos. 3 and 4 (1927), pp. 73 and 101.

[75] "Eenige beproevingsmethoden ter bepaling van den inwendigen wrijvingeweers tand van grondsorten," by A. S. Buisman, *De Ingenieur*, Vol. 26 (1928), p. 135 *et seq.*

[76] "Physical Properties of Earths," by John H. Griffith, *Bulletin 101*, Iowa Eng. Experiment Station, Ames, Iowa, 1931.

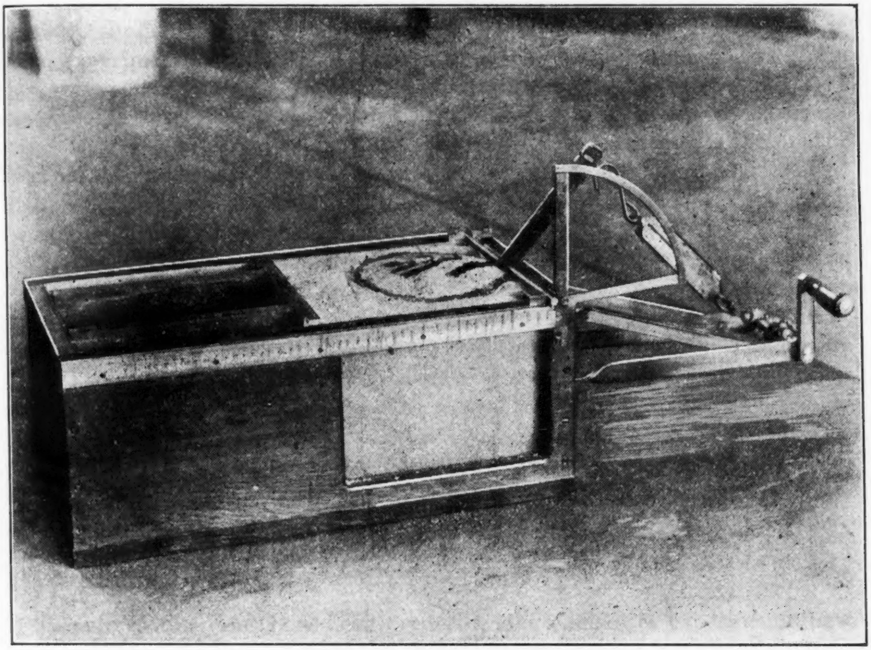

FIG. 30.—CONE OF MOIST OTTAWA SAND CREATED BY TURNING A METALLIC STAKE ABOUT A HORIZONTAL AXIS.

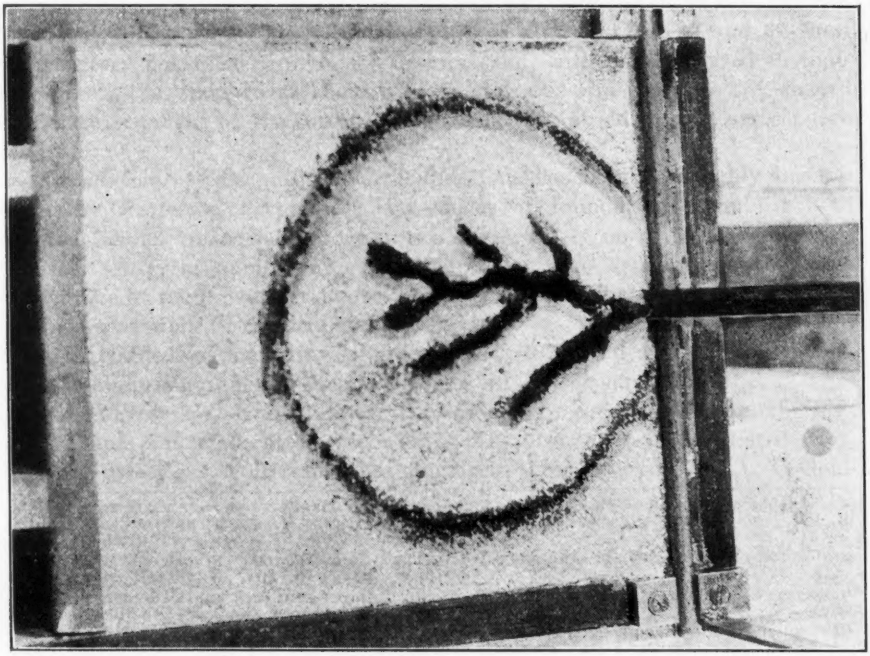

FIG. 31.—PLAN VIEW IN WHICH CHARCOAL INDICATES HOW NEARLY A TRUE CIRCLE IS FORMED.

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in 1931, similar tests made in Java, were described[77] by O. J. Herz. The Java experiments consisted in pulling some buried concrete masses from the ground by applying a vertical force. Both the Pittsburgh and the Java experiments disclose the possibility of determining shearing resistance of soil. In Fig. 28, MN is the surface of the ground; A represents the anchor used in the Pittsburgh experiments; F is a vertical pulling force measured with the dynamometer; and C is the surface of the cone of destruction (shearing surface).

In 1930 the writer proposed a field-stake method for determining shearing resistance of loamy soil. The method consists of driving a rigid metallic stake into the soil and pulling it with an automobile. In the beginning of the experiment the stake turns about a zero point, Z (Fig. 29). Cracks are then formed within a circle on the ground; and, finally, a conic body is pushed out. Let $S =$ reading on the scale (Fig. 29); $a =$ moment arm of the force, S, with respect to the point, B, which is the center of rotation of the stake when the cone is being pushed out; $r =$ the radius of the upper base of the cone; $W =$ the weight of the cone; and $R =$ the vertical resultant of the shearing resistance along the surface of the cone. Then, the condition of limit equilibrium would be:

$$S\ a = (R + W)\ r \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Simplified diagrams have been prepared, and a series of experiments have been performed in Connecticut. In more important cases the shearing resistance may be determined by applying an overturning moment to vertical piles or posts. The foregoing procedure does not represent an elaborate method, but rather discloses an idea requiring further study. Figs. 30 and 31 show a laboratory box filled with moist Ottawa sand, with charcoal used to indicate cracks. A stake turns about a horizontal axis and produces tree-shaped and circular cracks on the surface of the soil. The circle is quite perfect (see Fig. 31).

Foundations.—The question of foundations has been thoroughly investigated by Professor Terzaghi.[78] The statics of foundations should not be studied further in the laboratory unless a completely new line of research is evolved. A great number of experimental data are available, and a large percentage of these data are known not only to investigators, but to practical engineers as well. For instance, transmission of pressure through soils has been the subject of many investigations since 1879, and a more or less complete bibliography thereof may be found in a German publication.[79] Its authors, Professor Koegler and Dr. Scheidig, conducted comprehensive experiments and made attempts to develop a theory of pressure distribution. In some directions their results are similar to those obtained[80] by A. T. Gold-

[77] "Zugwiderstand eines Mastfundamentes und Scherfestigkeit des Lehmbodens," by O. J. Herz, *Zeitschrift* des Oesterreichischer Ingenieur und Architekten Vereins, Vol. 83 (1931), Nos. 9–14, pp. 59, 73, 93 *et seq.*

[78] "The Science of Foundations," by Charles Terzaghi, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 270.

[79] "Druckverteilung im Baugrunde," by F. Koegler and Dr. Scheidig. A series of articles published in *Bautechnik,* 1927–1929.

[80] *Proceedings*, Am. Soc. for Testing Materials, 1916, Pt. 2, p. 309, and 1917, pp. 640–641; *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 893; and *Public Roads*, Vol. 5 (1925), p. 1.

beck, M. Am. Soc. C. E. The "pressure bulbs" have also attracted the attention of engineers.

For this reason, possibly, no research work along these lines has been conducted in the laboratories of the Massachusetts Institute of Technology. A comprehensive theory of pressure distribution is still lacking, however; possibly it will be elaborated in connection with observations on a larger scale.

A few years ago, the idea arose of applying the Boussinesq formulas to foundation problems. These formulas, derived by applying the theeory of potential function,[81] have long been known to civil engineers; for instance, Professor S. Timoshenko discussed them before 1918. In 1928 the idea was formulated by Professor Koegler and Dr. Scheidig[79], and, in 1929, fully developed by Professor Terzaghi.[82] An endlessly wide and elastic body is assumed to be bounded by a horizontal plane above it (Fig. 32). A vertical load, P,

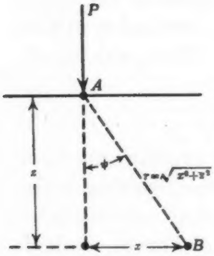

FIG. 32.

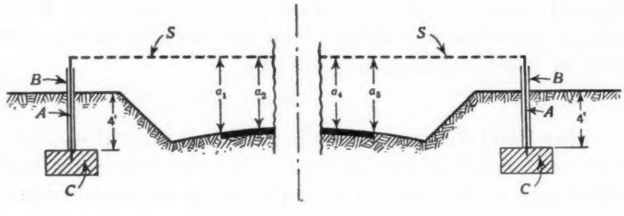

FIG. 33.—SAMPLE FIELD DEVICE FOR OBSERVING FROST HEAVING (NEW HAMPSHIRE).

is applied to a point A, on this plane. Under the action of the load, stresses develop in the interior of the body. One of the three principal stresses at the point, B (namely, the vertical), may be computed by means of the formula:

$$p_z = \frac{3\ P}{2\ \pi\ z^2}\ \cos^5\psi \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

If there are two or more concentrated loads, the computation should be made separately for each of them, and the results added, on the assumption that the stresses are superimposed. Equation (19) is simple, in itself, but in practice its application requires tiresome computations. Consolidation tests should be performed and theoretical data should be checked against actual observations of settlement.

Settlements plotted against time give a decreasing curve if regular consolidation takes place. This curve becomes a straight line (except the initial portion), if pure plastic flow exists. Finally, if both causes act simultaneously, a curve is obtained, which is rather difficult to analyze in the first year of a building's existence.

[81] "Application des potentiels à l'étude de l'equilibre et du mouvement des solides élastiques," by V. J. Boussinesq, Paris, 1885, p. 104 *et seq.*

[82] "Settlement of Buildings Due to Progressive Consolidation of Individual Strata", by Charles Terzaghi, M. Am. Soc. C. E., *Journal of Mathematics and Physics*, Vol. 8, No. 4 (1929), p. 266 *et seq.*

In this connection the necessity of obtaining good statistical material on settlements (as Professor Gilboy observes under the heading, "Foundations"), should be emphasized. At present, many superintendents and contractors consider even the possibility of settlement of their building as shameful; it should be borne in mind, however, that, as a rule, any building settles. The data on settlement should be surveyed and represented uniformly throughout the country. It would be better obviously, if the study of such settlements could be started on an international scale. Professor Terzaghi has collected numerous data along these lines in Europe, and the possibility of such an international study in close connection with him, deserves further consideration.

The subject of pile foundations has been well studied; especially valuable information has been presented[78] by Professor Terzaghi.[83] However, there is a branch of pile technique, somewhat different from foundations proper, that has scarcely been touched by research workers, namely, the stability of isolated piles and sheet-piles under the action of horizontal forces. Engineers require rules concerning the design of such features, but no adequate theory exists.

Hydraulic-Fill Dams.—It is rather difficult to discuss this important question so briefly outlined by the author in one and one-half pages of text. The research work described is more than interesting, and it is to be hoped that at least an abstract of the mathematical formulas in question will be published some time.

The writer believes that the path of hydrodynamical phenomena in the core should depend on the physical properties of the material of the shell; and this fact should be taken into account. In other words, the theoretical consolidation diagram (Fig. 12), which possibly has been traced for a certain homogeneous material, should vary according to the physical influence of the surrounding shell. This is because the shell forms a peculiar medium in which those hydrodynamical processes develop. If so, a new problem in soil physics is introduced.

Frost Heaving of Highways.—Generally speaking, the influence of low temperatures is not an important problem in foundation engineering. Exception should be made for permanently frozen soils, such as are met in Siberia and in some other localities. A vast literature on the subject exists in the Russian language; a short account in English may be found in a paper by the writer.[84]

The frost-heaving phenomena on highways have been thoroughly studied and are well understood by American scholars and engineers. For instance, Professor Stephen Taber, of the University of South Carolina, was the first to formulate the idea that the continuous supply of water from lower layers is responsible for heavings. The old theory assumed that frost heaving is due to change in the volume of water frozen in the voids; this theory was

[83] "Die Tragfestigkeit von Pfahlgründungen," by Charles Terzaghi, M. Am. Soc. C. E., *Die Bautechnik*, Vol. 8 (1930), No. 31, p. 475 *et seq.*; and No. 34, p. 517 *et seq.*

[84] "Soil Investigations in Russia," by D. P. Krynine, M. Am. Soc. C. E., *Proceedings*, Ninth Annual Meeting, Highway Research Board, 1929, p. 68 *et seq.*

based on experiments with closed systems[85] In applied research the work of the late Mr. V. R. Burton and of Mr. A. C. Benkelman should be mentioned.[86] Progressive research in Michigan tends to introduce some corrections in the foregoing theories. Tests have disclosed some soil textures that are known to be productive of the majority of severe frost disturbances in Michigan and in other States in the frost area.

A simple field device used in the central part of New Hampshire for observing frost heaving, is shown in Fig. 33. In this diagram, *A* is a metallic pipe, $\frac{7}{8}$ in. in diameter, embedded in a concrete block, *C*. There is also an outer pipe, *B*, 2 in. in diameter. The space between the two pipes is filled with grease. The block, *C*, is located 4 ft. below the earth's surface and is thus prevented from the action of the frost, since frost penetration is known to be less than 4 ft. The purpose of the outer pipe, *B*, is to insure the inner pipe, *A*, from any disturbance which may be caused by the frozen soil. A similar device is constructed on the other side of the road, and a horizontal string, *S*, may be drawn between them during the observation. The level of the string, *S*, is constant and may be considered as the datum plane. Vertical distances, a_1, a_2, a_3, etc., are recorded at predetermined points on the string every ten days during the winter. Thus, the rate of heaving may be determined. These experiments were begun under the supervision of Mr. Casagrande.

Retaining Walls.—Only a few years ago the theory of retaining walls constituted practically all the science now known as "soil mechanics."[87] Those who are attracted by the modern theory of a perfect fluid as acting on a retaining wall would possibly find new material for reflection in the descriptions of the tests cited by Professor Gilboy.[88] These tests demonstrate that, besides being characteristic of a given soil, internal friction is also indicative of its state of compactness, and that this varies within a wide range in the same structure.

Soil Mechanics Research Abroad.—As a rule, good scientific results may be reached when investigators are familiar with results obtained by others. Neither the branches of soil mechanics research in the United States, nor the names of American investigators have been specified completely in the preceding discussion. America is especially famous for its road and highway soil research. If, however, this country is far ahead of others in road soil research, its laboratories in some other branches are not as well equipped as some in Europe.

In the understanding of the writer, foreign countries in which soil mechanics research is being conducted may be divided into two categories: (*a*) Those with highly developed research, such as Germany, Austria, and

[85] "Frost Heaving," by Stephen Taber, *The Journal of Geology*, Vol. 37 (1929), No. 5, p. 428 *et seq.*; and "The Mechanics of Frost Heaving," by Stephen Taber, *The Journal of Geology*, Vol. 38 (1930), No. 4, p. 303 et seq.; see, also, other publications by the same author.

[86] "The Relation of Certain Frost Phenomena to the Subgrade," by V. R. Burton and A. C. Benkelman, *Proceedings*, Tenth Annual Meeting, Highway Research Board, National Research Bureau, 1930, p. 259 *et seq.*; see, also, many other publications by the same authors.

[87] "History of the Development of Lateral Earth Pressure Theories," by Jacob Feld, Assoc. M. Am. Soc. C. E., *Proceedings*, Brooklyn Engr.'s Club, January, 1928.

[88] *Engineering*, May 30 and June 13, 1930.

Sweden; and (*b*) those with less developed research, such as England and its Dominions, Russia, Poland, Holland, Hungary, Japan, and others. France has given to the world many original theories of "classical" soil mechanics but is not participating in modern soil research as based on soil physics.

Great Britain is especially noted for excellent research work on the physical properties of soil, including soil dynamics, carried on in the Rothamsted Agricultural Experiment Station.[45] Isolated papers from other English-speaking countries appear from time to time in the technical press. Russia has followed principally the American path of research; but has developed several interesting ideas of its own. The influence of the highly developed and unique Russian agricultural soil science was evident in the initial soil mechanics research in that country. Roads and highways form the principal province of activity for the Russian investigators, who would carry their work much further with better laboratories. Poland has also conducted its research along road lines. Holland works very actively in the field of engineering geology, which is closely allied to soil mechanics, and in soil mechanics itself, as may be seen from Dutch technical publications. Hungary has such an efficient investigator as Professor Janicsek. Japan has studied earthquake problems as applied to civil engineering, and some research on soil mechanics proper has been done as, for instance, the study of shearing resistance.[89] A special Geotechnical Committee has been organized in Japan. Some soil research work seems to have been done in South America, in the Argentine Republic, in connection with the problem of dirt roads.[90]

As far as the writer knows, some of the well-equipped laboratories in Austria, Germany, and Sweden are: (*a*) Technische Hochschule, Vienna, Austria; (*b*) Preussiche Versuchsanstalt für Wasserbau und Schiffsbau, Berlin, Germany; (*c*) Deutsche Gesellschaft für Bodenbaumechanik, Berlin, Germany; (*d*) Foundations and Hydraulics Institute, Hanover, Germany; and, (*e*) in Sweden, at the Technical University of Stockholm, and the Swedish Bureau of Public Roads.

In establishing the Laboratory of Soil Mechanics at the Technische Hochschule, in Vienna, Professor Terzaghi perfected the actual research equipment and developed new methods. The Austrian Government made a considerable appropriation for its installation.

At the "Preussische Versuchsanstalt für Wasserbau und Schiffsbau," in Berlin, special emphasis has been laid on soil mechanics research in the Department of Foundation Engineering.

The laboratory of the German Society for the Development of Soil Mechanics ("Deutsche Gesellschaft für Bodenbaumechanik") was formed by governmental engineers, with headquarters at the Technische Hochschule, in Charlottenburg, Berlin. Its principal aim is the study of emergencies in connection with earth structures (water-works failure, landslides, embankment failures, etc.).

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[89] "'Shearing Resistance of Soils with Various Constituents," by N. Yamaguti, *Bulletin*, Japanese Govt. Railways, Vol. 19 (1931), Nos. 1 and 25, in Japanese.

[90] "Construcción de Caminos en Paises Nuevos," by Roberto Kurtz, Assoc. M. Am. Soc. C. E., *Proceedings*, Sixth International Road Congress, Washington, D. C., 1930.

The Soil Mechanics Laboratory of the Foundations and Hydraulics Institute, in Hanover, has a separate new building provided with good equipment; for example, a huge retaining wall is a part of it. The dimensions of the wall are about the same as those of the apparatus in the Massachusetts Institute of Technology, as described by Professor Gilboy.

Among the Swedish establishments there are two small separate laboratories of soil mechanics at the Technical University of Stockholm, and a small laboratory of the Goetechnical Commission of the Swedish Railroads. The soils laboratory of the Swedish Bureau of Public Roads ("Svenska Väginstitutet"), maintains an active contact with geologists, and there are members of the Swedish Geological Survey who have specialized in the problems of both soil physics and road engineering, approaching the latter, of course, from the geological point of view.[91]

J. STUART CRANDALL,[92] M. AM. SOC. C. E. (by letter).[92a]—This paper gives an excellent résumé of some of the progress made during the last few years in determining the nature and physical characteristics of soils. The importance to the engineer of these developments is apparent, because all structures must be supported by the soil through some type of foundation. The rate of settlement of these foundations over a period of years is important, and occasionally vital. Soil mechanics is a connecting link between geology and civil engineering. Further and more rapid advances may confidently be expected in the future. It must be admitted that, on the whole, the technique of foundation design has been rather empirical, with little actual rational basis. There is need for change. It is to be hoped that the profession may sincerely seek logically to apply the results of these studies to actual foundation design.

Most building codes are very definite as to the allowable load on different soils, but are rather vague as to the classification of these soils. The author might have pointed out the objectionableness of the present common practice of leaving to the foreman of a boring crew the identification of the type of soil under a foundation. He might have pointed out also the common lack of attention and care to preserve boring samples in the natural state. Frequently, soils are classified as "clay," when the material is actually silt. This occurred recently at Rochester, N. Y., where borings were taken at the site of the Charlotte Terminal at the mouth of the Genesee River. The boring foreman identified the underlying soil as "soft blue clay" and "medium soft gray clay." The proposed quay wall was of the L-type of reinforced concrete supported by wood piles, with a steel, sheet-pile bulkhead along the face, tied back by rods attached to a back-wall of steel piling. Analysis of the entire structure indicated a small factor of safety against rotation, on account of the low angle of internal friction and low shearing resistance of clay. Samples from the borings were then analyzed, which indicated that the material was not clay, but silt with a small admixture of clay. The coefficient of internal

[91] Svenska Väginstitutet, Meddelanden 13; 15 (1929); 21; 24; 25; 26 (1930); 30 (1931), av Dr. G. Bescow, Stockholm.

[92] Vice-Pres. and Engr., The Crandall Eng. Co., Cambridge, Mass.

[92a] Received by the Secretary December 31, 1931.

friction and shearing resistance of the silt being much higher than clay, there was no longer any question of stability. Such occurrences show the necessity for the adoption of a reliable and accurate method of classifying the type and condition of soils.

Of great importance is the very evident fact that for cohesive, impervious soils, such as clay, the usual load-settlement tests required by various building codes are of little value, for within the usual short duration of such tests there is not sufficient time to determine the actual rate of settlement. Furthermore, the zone and intensity of stress in the soil directly under the small area loaded are very different from the zone and intensity of stress under the entire foundation.

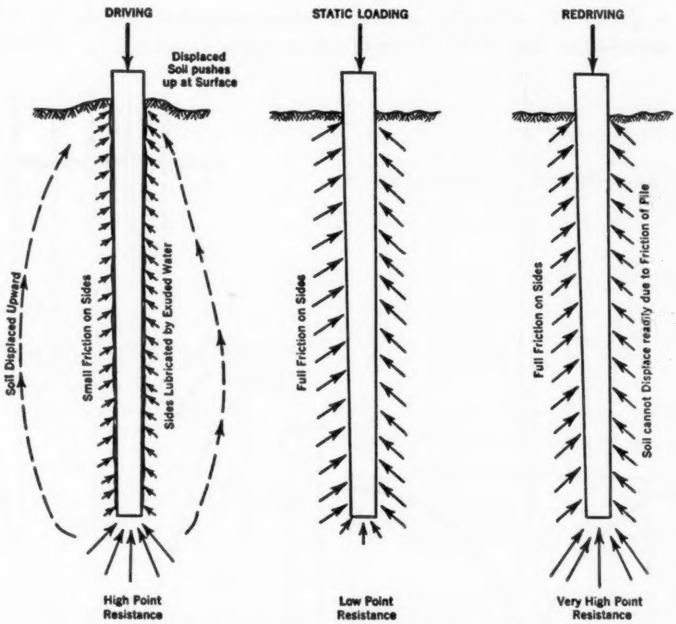

FIG. 34.—PILES IN SILT, CLAY, ETC.

Clay being practically incompressible over short periods of time, it is evident that the driving of piles cannot act to consolidate it; witness the swelling of the surface when piles are driven in clay. This being the case, the very work of driving must disturb and knead the clay, and thus increase the rate of settlement. It is logical to assume, therefore, that the driving of piles into clay should be avoided, and that it is better practice in such soils to sink caissons or caisson piles so that there may be little disturbance and so that the foundation will bear on the material in its natural state.

Charles Terzaghi, M. Am. Soc. C. E., has demonstrated[93] that there is no known relationship between the dynamic resistance of piles driven in clay and

[93] "The Science of Foundations—Its Present and Future," *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 270.

the resistance to static load. In Fig. 34 is shown another way of illustrating the causes of this phenomenon.

This being true, the attempt to determine the bearing capacity of piles driven in clay by the use of some pile-driving formula has no logical or scientific basis and cannot give results of any value. Load tests of individual piles give no indication of the settlement of a group of piles with the same load per pile. The approximate distribution of the vertical component of stress in the soil at the plane of the point of a single pile is shown in Fig. 35.

For a group of piles, the zones of stress overlap, as shown in Fig. 36 (which represents the case of twenty-five piles spaced at a distance equal

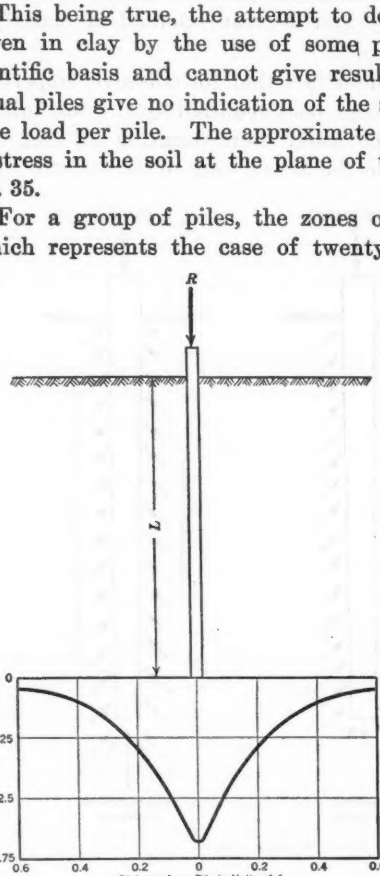

FIG. 35.—APPROXIMATE VERTICAL STRESS IN SOIL AT PLANE OF PILE POINTS, INDIVIDUAL FRICTION PILE.

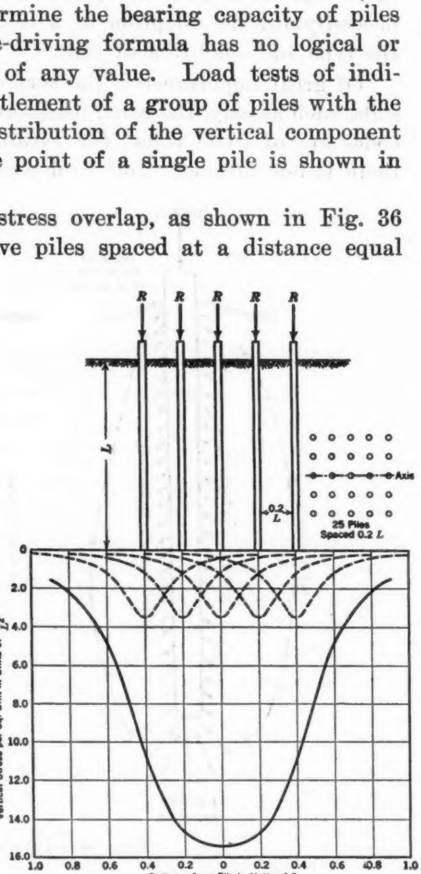

FIG. 36.—APPROXIMATE VERTICAL STRESS IN SOIL AT PLANE OF PILE POINTS, GROUP OF FRICTION PILES.

to $0.2L$), causing a stress in the soil nearly five times that due to a single pile with the same load. In such cases, it would seem to be better practice to determine analytically the stress in the soil along the plane of the points, compare this with the bearing resistance of the soil, due allowance being made for probable settlement based on an analysis of the underlying material, and then, by trial and error, calculate the most economical length, spacing, and load per pile.

It is to be hoped that from time to time the author may present further reports of progress in this relatively new and important field; and it may be further hoped that engineers will co-operate to the fullest extent in formulating procedure for the practical application of the discoveries in soil mechanics to actual engineering problems.

FRANK ALWYN MARSTON,[94] M. AM. SOC. C. E. (by letter)[94a]—Substantial progress was made during the five years, 1927 to 1932, in devising methods of determining the physical properties of soils. The practical application, however, of data concerning these physical properties to studies of the behavior of soil in engineering structures is somewhat less advanced. This paper is, therefore, a valuable contribution, because it gives in brief form a description of certain tests which have been found to be of distinct aid to judgment in passing on the suitability of soils for the building of dams and embankments and the solving of foundation problems.

The opportunities for research are great. Programs of scientific study such as are being carried on in various institutions of learning are most necessary for the development of methods of analyzing soils. Few laboratories outside of such institutions have the opportunity or the personnel that will permit of extensive research. To the work of the scientific laboratory must be added the opportunity to make practical application of the results of research by investigations in the field on full-sized structures, if tangible results are to be obtained.

Soil physics and soil engineering are destined to become of greater importance as time goes on because it will be more generally recognized that judgment aided by scientific soil analyses may be more dependable than the unaided judgment of even the experienced engineer.

TABLE 1.—A COMPARISON OF SOME TYPICAL FINE-GRAINED SOILS

Source of sample	Percentage by Weight of Grains of Size Less Than			Liquid limit, percentage*	Plasticity index†	Angle of internal friction‡	Sample number	Percentage of organic matter
	0.1 mm.	0.01 mm.	0.002 mm.					
Sewer trench, Pawtucket, R. I.	97	24	0	20.7	0	35°–10′	2 145	....
China clay	100	78	40	52.2	11.8	......	A-28	....
Core, Alexander Dam, Kauai, Hawaii§	94	53	9	75.2	11.8	37°–57′	A-23	10.6
Marine mud, Boston, Mass.	98	55	18	78.9	42.0	19°–16′	A-2	7.3
Blue gray clay, Port Arthur, Tex.	100	87	61	93.7	59.1	9°–39′	A-11	....
Fossiliferous earth, Springfield, Ohio	89	28	9	118.8	71.7	......	A-32	20.1

* Ratio of weight of water to weight of dry solids expressed in percentage.
† Difference between liquid and plastic limits.
‡ Determined from shearing resistance.
§ Contained 64.8% of iron as Fe_2O_3.

Separation of the soil sample into fractions according to grain size is helpful, but if the analysis stops there, conclusions may be misleading. Table 1 gives data relating to several samples of soil in each of which nearly all the grains were less than 0.1 mm. in size, that is, of a size approximately, that would pass the 150-mesh sieve. On the basis of the sieve analysis, the samples are more or less comparable. The fine fractions, determined by sedimentation, vary widely. The china clay, containing 40% by weight of grains less than 0.002 mm. in size, has a low index of plasticity, whereas the fossiliferous earth with only 9% finer than 0.002 mm. has a plasticity index of 71.7. Gen-

[94] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[94a] Received by the Secretary January 12, 1932.

erally, the larger the plasticity index the more plastic and slippery is the material. The index may be considered as a rough measure of the cohesion existing between the soil particles. A high index may indicate high compressibility and slow rate of consolidation. Non-plastic soils have an index of zero. It is evident from the examples given, that the percentage content of fine particles may not be a controlling factor in the relative behavior of soils from various sources. On the other hand, with a given soil, the proportion of fine particles may have a strong influence on the behavior of the composite sample.[95]

The mechanical analysis is useful, but should be supplemented by tests in order to determine other characteristics of the soil, as pointed out by Professor Gilboy.

After some studies of the results of analyses by the Wiegner and Bouyoucos sedimentation methods, the writer is of the opinion that the Bouyoucos

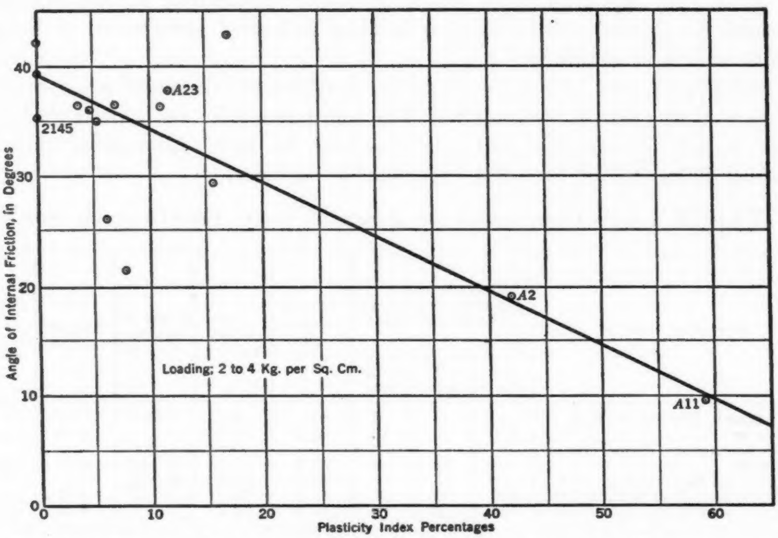

FIG 37.—TYPICAL RELATION BETWEEN ANGLE OF INTERNAL FRICTION AND PLASTICITY INDEX.

method is the more convenient, and when conducted according to the procedure developed by the U. S. Bureau of Public Roads, is more likely to give reliable results.

The Atterberg limits of consistency—liquid and plastic limits—are easily determined, and they furnish a valuable index to the plasticity of the soil. The liquid limit device has been used for some time and has been found to be superior to the hand method because more consistent results can be obtained and there is less variation due to the personal equation.

The interrelation of soil characteristics is such that it is difficult to select two characteristics and show that one is a function of the other. With a

[95] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 354, Table 9.

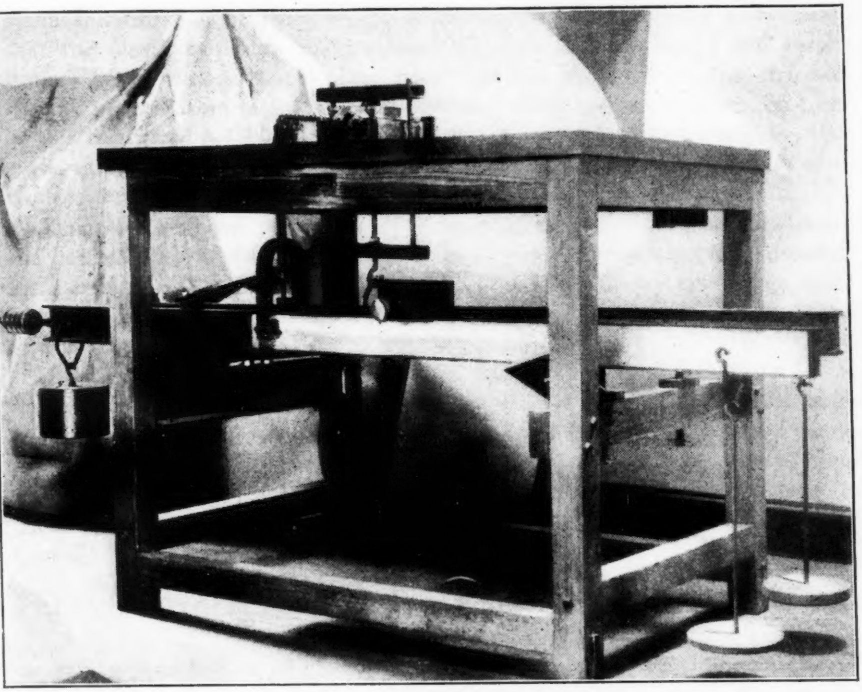

Fig. 38.—View of Shearing Machine.

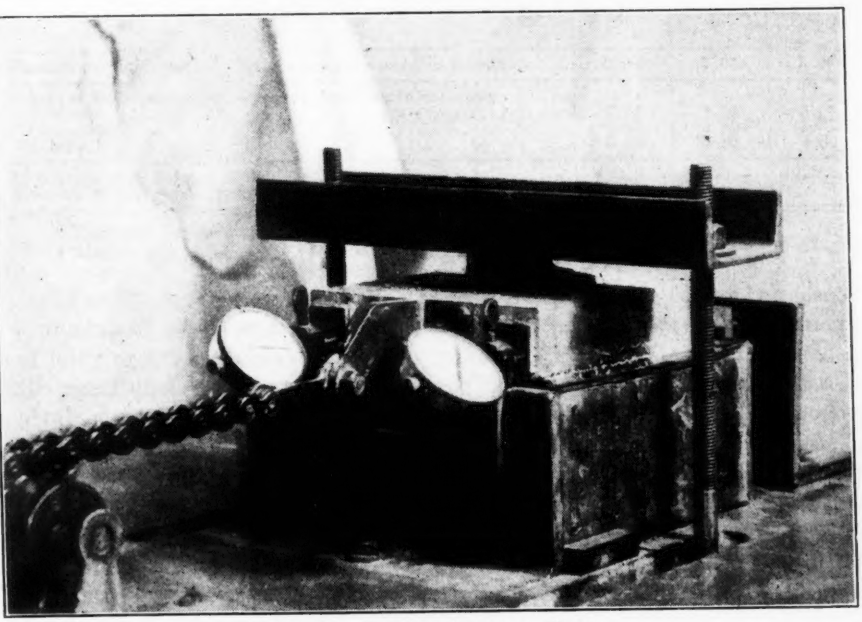

Fig. 39.—View of Shearing Unit.

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given soil this can be more easily demonstrated. Comparisons have shown that the plasticity index (percentage difference between liquid and plastic limits) bears some relation to cohesion and the angle of internal friction. Fig. 37 plotted from test data representing a variety of soils shows the trend of this relationship. The numbered points refer to samples in Table 1. Such a diagram can be used as a preliminary rough guide especially when compiled from materials from one locality.

The variable-head permeameter is a most useful apparatus for testing the permeability of fine-grained soils. It is easy to use and gives reasonably consistent results. For very impervious soils, the permeability may be determined from the consolidation test, but this requires more elaborate apparatus and a longer time.

Studies of earth dams, embankments, and earth cuts require data on the angle of internal friction and the cohesion of the soil. Determinations based on the compressive strength of clay cylinders are interesting, but the results are erratic. In some cases the angle of internal friction computed from the observed angle of rupture agrees fairly well with that determined from the shearing resistance test and, in other cases, considerable variation has been found.

TABLE 2.—Typical Test of Shearing Resistance—Sample Pulled Back After Each Slip

Test characteristic	Yellow Sandy Clay:† Tests		
	A	*B*	*C*
Pressure per unit area*, in kilogrammes per square centimeter	0.93	3.03	5.04
Shear per Unit Area, in Kilogrammes per Square Centimeter:			
First slip	0.46	1.76	3.56
Second slip	0.55	1.89	3.71
Third slip	0.59	2.03	3.60
Moisture content at start, percentage	23.9	....	....
Moisture content at end, percentage	15.7	....	....

* Time for consolidation allowed after each change in loading.

† Sample saturated with water during test.

The cylinder rupture method can be performed quickly and can be repeated a number of times on many samples without involving a prohibitive amount of labor or time, whereas only a few representative samples can be tested in the shearing resistance machine. Using both methods, information can be obtained from a large number of samples, especially if they are grouped according to the plasticity index or some other standard or characteristic.

After the first shearing device, referred to by Professor Gilboy, was built, another machine was designed following the principles of the first machine, but with fewer refinements. A general view of the apparatus and a detail view of the shearing device are shown in Figs. 38 and 39. At first, a double wooden frame was used to contain the soil sample, but, later, this was changed for one made of aluminum.

An interesting series of experiments have been tried by pulling the upper half of the frame back, after one shear test has been made, without removing the load, in order that an additional test may be conducted without necessitating the removal and replacing of the sample. The movement in each test was about $\frac{1}{2}$ in. forward, then $\frac{1}{2}$ in. back, then forward again, then back to the original position, etc. In this manner three shear tests were made with a minimum of horizontal movement. The moving back and forth seemed to effect a slight consolidation of the material, because the shear per unit area was generally higher on the second and third tests than on the first, but not always so. Table 2 is typical of the results of such a procedure. The data are of interest as showing the prompt re-adjustment of the soil particles after a slip.

It is important that sufficient time be allowed after loading the soil for consolidation to take place. With a cohesive soil the time required may be from one to two days. After that, the change is comparatively small. What happens over a period of months or years under constant load is not known,

TABLE 3.—Effect of Time of Consolidation Under Load upon Shearing Resistance

Test data	Brownish sandy clay*
Pressure per unit area, in kilogrammes per square centimeter	3.01
Shear per Unit Area, in Kilogrammes per Square Centimeter (Time, in Hours, after Placing Load):	
22	1.97 2.30
42	2.55 2.55
119	2.55
284	2.63
454	2.63
625	2.46

* Sample saturated with water during test.

but it seems probable that some further consolidation or increase in cohesion takes place, resulting in increased shearing resistance. Table 3 shows the results of a shearing resistance test covering 26 days. There was very little change in resistance after 42 hours, and these differences may be within the limits of error of the test.

In applying the methods described by Professor Gilboy to studies of soils for a rolled earth dam, it is not so necessary to use samples of undisturbed soil. Material in the borrow-pit is excavated and then deposited in the dam in thin layers which are thoroughly rolled. Much the same process can be carried out on a small scale in the laboratory. Undisturbed samples should be taken from the dam from time to time to check the degree of consolidation, moisture content, etc.

A helpful basis of study can be had by comparing the void-ratio of undisturbed samples of the embankment with the void-ratio of laboratory compacted samples. In general, the field-compacted samples are likely to have a little larger void-ratio than samples thoroughly rammed, kneaded into compact form, and similarly loaded in the laboratory.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

CONSTRUCTION WORK ON A FEDERAL RECLAMATION PROJECT

Discussion

By Messrs. Clifford A. Betts and George C. Imrie

Clifford A. Betts,[9] M. Am. Soc. C. E. (by letter).[9a]—The description of the scientific control of concrete production and of the up-to-date construction methods and costs on the Kittitas Division of the Yakima Project draws attention to the economies that can be effected by the use of laboratory control and modern machinery. There is a well defined trend in this direction.

The uniformity that can be assured in concrete by effective control permits closer design, lower safety factors, and correspondingly greater savings. On projects involving large yardages, the net savings from this source alone have amounted to as much as 2% of the total cost of the concrete, as evidenced by one case in which the cost of laboratory control and inspection was $0.035 and the saving in cement $0.18 per cu. yd. of concrete. Costly control becomes burdensome; but practical inspection administered with common sense can be made indispensable. Furthermore, on work that is being done on a three-shift program, it becomes highly advantageous to make the proportioning, mixing, and placing as "fool-proof" as possible by standardization of all operations.

In attaining the impermeability, strength, and workability required for long-lived hydraulic structures the following procedure has been found to have merit:

(*a*) Screen both sand and gravel into two or more sizes in order that the screened aggregates may be reblended accurately in such a manner as to produce concrete with a minimum percentage of voids. In this way a troublesome variation in consistency—which inevitably adds to the difficulties of water control and intermittently causes harshness, with water rising to the surface—can be avoided and a satisfactory gradation, which is a vital element, can be obtained.

Note.—The paper by Morris Mason, Jun. Am. Soc. C. E., was published in October, 1931, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: December, 1931, by John Sanford Peck, Assoc. M. Am. Soc. C. E.; and January, 1932, by Messrs. Edward W. Bush and Orrin H. Pilkey.

[9] Engr., U. S. Bureau of Reclamation, Owyhee Dam. Nyssa, Ore.

[9a] Received by the Secretary January 4, 1932.

(*b*) Locate washing and screening plant and arrange the stock-piles so that aggregates will not carry a wide variation of moisture into the mixer bins to interfere with moisture control.

(*c*) Proportion by weight, with graphic corrections for surface moisture and carry-over, the latter requiring attention only when screens are changed. The use of charts at the mixer eliminates interpolation and reduces computations to the subtraction of the surface moisture from the mixing water.

In Fig. 21, a typical illustration, the proportions of each size of aggregate are expressed in terms of parts by weight of cement per batch (diagonal lines marked "stock-pile proportions"). The percentages of surface moisture

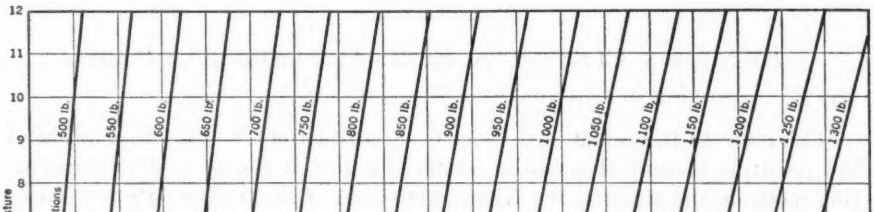

FIG. 21.—WEIGHTS OF AGGREGATES FOR 1 CUBIC YARD BATCH.

are ordinates and the weights of aggregates plus moisture, are abscissas. For example, when the surface moisture of a certain aggregate is 5% and the specified proportions are 2 parts to 1 part of cement, the total weight of this moist aggregate required for 1 cu. yd. batch is 1 050 lb. Similarly, the quantity of water to be deducted from the mixing water (in gallons or pounds) can be read from a diagram, such as Fig. 22, having "parts of aggregate by weight" as ordinate, "water to be deducted from total mixing water" as abscissa, and "percentage of surface moisture" as diagonals, with angles of inclination as determined in the laboratory. As an example, if sand contains 4% moisture, the chart shows that 4.8 gal. of mixing water should be deducted per batch.

In addition to the methods of surface moisture determinations described in the paper, reference might be made to patented scales reading the percentage of moisture directly on the dial.

Such control applied to a job of sufficient yardage to give a large number of representative samples will permit strength control tolerance so close that no specimens will vary more than 15% above or below the average of all the samples taken, and 90% will be within 10% of the average. As a rule, works involving considerable concrete yardage and extending over several years, show a gradual improvement in quality of concrete produced, as time goes on. The better the control the more nearly all the concrete will approach the perfected product.

The statement by Mr. Morris that "high strength, high impermeability, and good workability * * * were considered equally important," is a

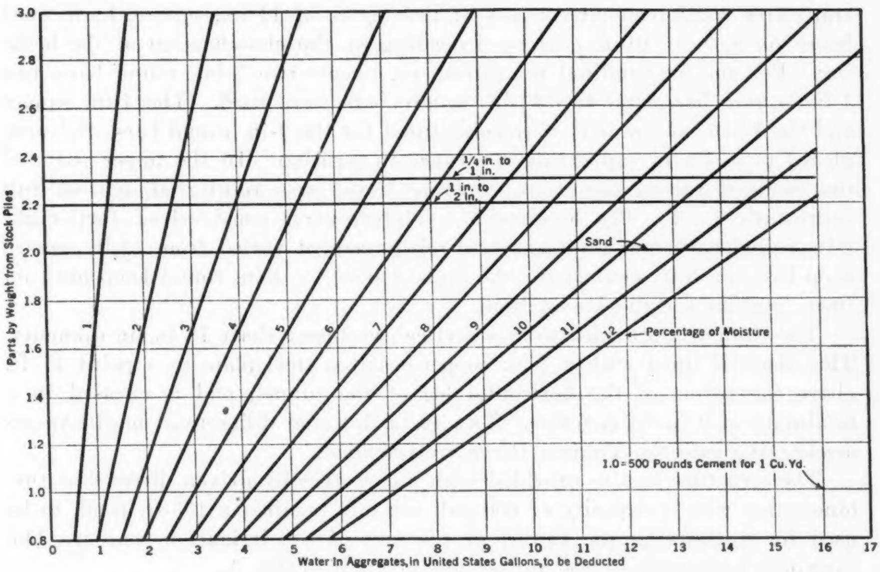

FIG. 22.—SURFACE MOISTURE IN AGGREGATES, 1 CUBIC YARD BATCH.

reminder that a standard test for impermeability is greatly needed. In fact, it frequently happens in hydraulic structures that water-tightness rather than strength is the criterion controlling the richness of the mix. Unfortunately, there is no well-defined quantitative relation between strength, which is readily tested, and impermeability, which is more difficult to measure, although, in general, both are improved by the same factors. For example, placing and curing, like gradation, have a greater effect on permeability than on strength, whereas wetting a mix may decrease the strength, but increase imperviousness. Pending the perfection of a generally acceptable permeability test apparatus a simple qualitative test can be made by inserting flanged pipes into mass concrete and determining by trial the permissible hydrostatic head.

GEORGE C. IMRIE,[10] ESQ. (by letter).[10a]—Taken as a whole this paper gives a true picture of the construction work as conducted on the Kittitas Division of the Yakima Project. Because of conditions encountered as the work progressed, some major changes were made in the construction program. In driving the horizontal leg of the tunnel several sections were excavated through tuffaceous shale, and it was decided to change the type of lining through this part of the tunnel from plain to reinforced concrete, and to increase the thickness of the shell from 15 to 30 in. Where the tunnel was excavated through a substantial covering of basalt, it was lined with 15 in. of plain concrete as covered in the specifications.

In the 30-in. reinforced sections of the horizontal leg of the tunnel, the transverse reinforcement consists of two layers of $1\frac{1}{2}$-in., square, hard, steel hoops on 8, 9, or 10-in. centers, depending on the classification of the backing. For the longitudinal reinforcement, twenty-two $\frac{3}{4}$-in., round bars, two 1-in. square bars, and two $1\frac{1}{4}$-in. square bars were used. The 1-in. square and the $1\frac{1}{4}$-in. square bars were substituted for the $\frac{3}{4}$-in. round bars, and were placed to assist in supporting the cages in position. In the upper parts of the inclined raises, the 15-in. concrete lining was reinforced against full hydrostatic head. The longitudinal reinforcement consisted of forty-eight $\frac{5}{8}$-in. round bars, and the transverse reinforcement varied from $1\frac{1}{2}$-in. square hoop bars, on 6-in. centers for the highest head, to 1-in. round hoop bars, on 10-in. centers for the lowest head.

The tunnel is provided with a vertical wasteway shaft 78 in. in diameter. This shaft is lined with a $\frac{3}{4}$-in. copper-bearing, steel plate to a point 17 ft. above the invert of the horizontal leg of the tunnel, and is encased in a minimum of 9 in. of concrete. Two 42-in. internal differential needle-valves discharging into the Yakima River are provided.

The structure is also provided with an electrically driven direct-flow turbine pump with a capacity of 600 gal. per min. against a 155-ft. head, to be used for unwatering the tunnel at the end of the irrigation season. The hydraulic properties of the tunnel are given in Table 3.

Of the 6 750 acres of land shown in the paper as held by the Northern Pacific Railway Company, all but 639 acres have now (February, 1932) been sold at the appraised value to *bona fide* settlers.

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[10] Office Engr., U. S. Bureau of Reclamation, Ellensburg, Wash.

[10a] Received by the Secretary February 24, 1932.

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DISCUSSIONS

WATER PRESSURES ON DAMS DURING EARTHQUAKES

Discussion

BY MESSRS. R. ROBINSON ROWE, L. PRANDTL, AND CECIL E. PEARCE

R. ROBINSON ROWE,[14] M. AM. SOC. C. E. (by letter).[14a]—The most useful and interesting theory for the design of structures to resist earthquakes, that has appeared in recent years, is given in this paper. It establishes a criterion applicable to existing as well as to proposed dams.

Although the author does not state positively that his solution is unique, the three differential equations ((1), (2), and (5)) contain only three unknowns other than the dimensional variables, and should lead to a single primitive, made definite by the rigid boundary conditions. The latter should include the condition that ξ and η approach zero as x becomes large. Furthermore, Condition (3) should be modified to exclude the point of singularity; the condition is not met at that point, since Equation (21) is not true for $u = 0$.

That the solution is reasonable can well be judged by a study of the continuity of the dynamic action in the water, as shown graphically herewith. In each of the diagrams assumed the action is shown in the field, $0 < x$ (or y) $< h$, with $c_n = 1$, $T = \frac{4}{3}$, and $\alpha = 0.1$. No corrections have been made for phase difference.

In addition to the author's notation, the following are introduced: $u = \dfrac{\pi\, y}{2\, h}$; $z = e^{\frac{\pi x}{2h}}$, $\delta = \sqrt{\xi^2 + \eta^2}$ equals the displacement of a particle in the direction of the action; $\theta = \tan^{-1} - \dfrac{n}{\xi}$ equals the angle between the line of action and the horizontal; and max. is a subscript indicating the maximum value of the variable.

NOTE.—The paper by H. M. Westergaard, M. Am. Soc. C. E., was published in November, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings* as follows: February, 1932, by Messrs. Theodor von Karman and Paul Bauman.

[14] Civ. Engr. (Allen & Rowe), San Diego, Calif.

[14a] Received by the Secretary January 15, 1932.

Fig. 7 shows lines of equal horizontal displacement for the field, the values shown being maxima for $\alpha = 0.1$ and $T = \frac{4}{3}$ sec., corresponding to $t = \frac{T}{2}, \frac{3\,T}{2}$, etc. From Equation (15), $\xi_{max.} = 0.1846 \sum_{1,3,5,..}^{n} \frac{\sin\ nu}{n\ z^n}$; and $\xi = 0$ when $x = \infty$ or $y = 0$. When $x = 0$ and $y > 0$, $\xi_{max.} = 0.1460$; and when $y = h$, $\xi_{max.} = 0.1846 \cot^{-1} z$.

The curves indicate a gradual change in horizontal displacements, which is more rapid in the vicinity of the top of the dam, where the lines converge at the singular point. For $y = 0$ and $x < r$, the direct interference of the dam would make $\xi_{max.} > (r - x)$.

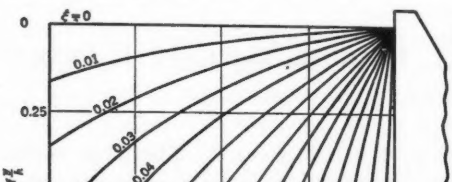

FIG. 7.—LINES OF EQUAL HORIZONTAL DISPLACEMENT

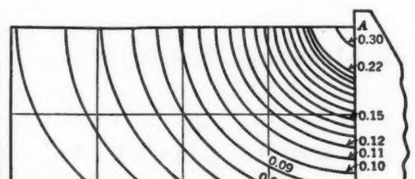

FIG. 8.—LINES OF EQUAL VERTICAL DISPLACEMENT.

Similarly, Fig. 8 shows lines of equal vertical displacement, with conditions the same as in Fig. 7. From Equation (16),

$$\eta_{max.} = -\,0.1846 \sum_{1,3,5,..}^{n} \frac{\cos\ nu}{n\ z^{-n}}$$

and $\eta = 0$ when $x = \infty$, or $y = h$. When $x = 0$,

$$\eta_{max.} = 0.09295 \log_e \tan \frac{u}{2}$$

and when $y = 0$,

$$\eta_{max.} = 0.09295 \log_e \left[\left(\frac{z-1}{z+1}\right)\right]$$

At Point A in Fig. 8, $\eta = \infty$.

As in Fig. 7, a gradual change in vertical displacements is evident, the change being more rapid in the vicinity of the top of the dam and infinite at the singular point. The lines are approximately normal to those of Fig. 7

throughout the field. Direct interference of the dam upsets the formula for $y = 0$ and $x < r$.

Fig. 9 shows line of equal displacements in the direction of the action, being the resultant of values shown in Figs. 7 and 8, since the horizontal and vertical displacements are in the same phase. The conditions are the same as for Fig. 7; and $\delta = \infty$ at Point A. Furthermore, $\delta = 0$ when $x = \infty$; when $x = 0$,

$$\delta_{\text{max.}} = 0.1460 \sqrt{1 + 0.4053 \left(\log_e \tan \frac{u}{2}\right)^2}$$

when $y = 0$,

$$\delta_{max.} = 0.09295 \log_e \frac{z - 1}{z + 1}$$

and when $y = h$,

$$\delta_{\text{max.}} = 0.1846 \cot^{-1} z$$

When x is large, all terms of the summations except the first are negligible, and $\xi_{\text{max.}} = 0.1846 \dfrac{\sin u}{z}$; $\eta_{\text{max.}} = -0.1846 \dfrac{\cos u}{z}$; and $\delta_{\text{max.}} = \dfrac{0.1846}{z}$. Since z is independent of y, the lines of equal displacement approach vertical straight lines. For the singular point, $\delta_{\text{max.}} = \infty$, of course.

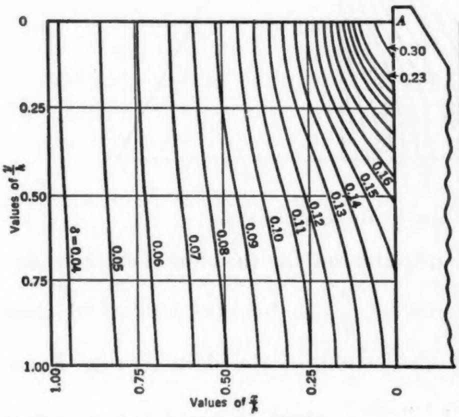

FIG 9.—LINES OF EQUAL DISPLACEMENT.

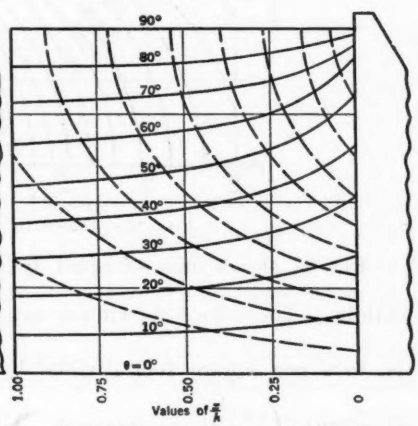

FIG. 10.—LINES OF ESCAPE AND OF EQUAL SLOPE

Dividing Equation (16) by Equation (15), $\tan \theta = F(x, y)$, and is independent of t. For any point, therefore, the slope angle of the displacement is constant throughout the cycle, and each particle moves in simple harmonic motion along an essentially straight line of length, $2\,\delta$, and inclined up and away from the dam, making an angle of θ with the horizontal. The solid curves in Fig. 10 are loci of equal inclinations; the action is vertical at the water surface and horizontal at the bottom. As in Fig. 7, c_n (assumed) $= 1$, and $T = \frac{4}{3}$ sec. When $x = 0$, $\tan \theta = 0.6366 \log_e \tan \dfrac{u}{2}$. When $x = \infty$, $\theta = u$.

The broken curves indicate lines of escape; that is, they are drawn tangent to the inclinations, and indicate the direction of the action. Since there is no motion across these lines, the volume of water between two such curves can be considered as acting independently. It is evident, therefore, that about 80% of the impulse finds surface escape within the limits of the field.

In this connection, the writer suggested in 1928 that the thrust due to earthquakes should not exceed that which would be caused by an assumed translation along circular quadrants centering at A (Fig. 9). With such an assumption, the surface escape within the field is 100%, and the following are easily computed: $p = 0.00491\ y$; $p_0 = 0.00491\ h$; $Q_0 = 0.00245\ h^2$; and $M_0 = 0.000818\ h^3$. The author's results for the last three, for an 800-ft. dam, are less by 44.6%, 20.1%, and 5.0%, respectively.

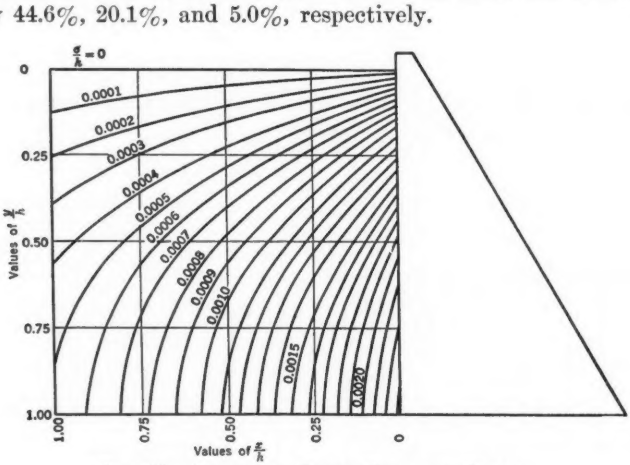

FIG. 11.—LINES OF EQUAL DYNAMIC STRESS.

Fig. 11 shows lines of equal dynamic stresses for the same field and conditions. The values shown are maxima for $\frac{\sigma}{h}$, in tons per cubic foot; that is, tons per square foot divided by the height of the dam. From Equation (19), $\left(\frac{\sigma}{h}\right)_{\text{max.}} = 0.002533 \sum_{1,3,5,..}^{n} \frac{\sin nu}{n^2 z^n}$, which is zero when $x = \infty$ or $y = 0$. When $y = h$, $\left(\frac{\sigma}{h}\right)_{\text{max.}} = 0.002533 \sum_{1,3,5,..}^{n} \frac{(-1)^{\frac{n-1}{2}}}{n^2 z^n}$; when $x = 0$, $\left(\frac{\sigma}{h}\right)_{\text{max.}} = 0.002533 \sum_{1,3,5,..}^{n} \frac{\sin nu}{n^2}$, which reduces to the series,

$$0.001267\ u\ \left(1 - \log_e \frac{u}{2} - \frac{u^2}{36} - \frac{7\ u^4}{7\ 200} - \frac{31\ u^6}{181\ 440} \ldots.\right)$$

The change in stress is gradual and continuous, without singularity.

The curves do not represent true pictures of instantaneous conditions, because phase differences have been disregarded. When maxima occur at $x = \frac{h}{2}$, the angular phase difference for $x = 0$, or $x = h$, is $\frac{2\pi t}{T}$, in which, $t = \frac{h}{2 v_s}$, so that the difference amounts to only $0.0004994\,h$. The distortion of the field at that time amounts to 0.5% for $h = 200$ and 7.9% for $h = 800$. Such distortion does not materially affect the representation intended.

It should be noted that the elastic deformation of the dam and the motion of the bed of the reservoir tend to diminish the stresses, and add to the security of the formulas. The possibility that the dam might be at the focus for compression waves set up in the water by the bed of the reservoir, seems to be exploded, in view of the large percentage of the impulse which escapes to the surface in a relatively short distance.

PROFESSOR DR. L. PRANDTL[15] (by letter).[15a]—In connection with this paper by Professor Westergaard, it might be of interest to note that this same mathematical problem is found in the theory of the distribution of lift on an aerofoil that is extended through an air stream of limited width and unlimited height. The water level in the problem by Professor Westergaard corresponds to the free boundary of the jet, and the bottom of the water tank to the middle plane of the jet. The calculations, that lead to exactly the same formulas, are contained in a paper[16] by Joseph Stüper, entitled "Der durch einen Freistrahl hindurchgesteckte Tragflügel."

CECIL E. PEARCE,[17] ASSOC. M. SOC. C. E. (by letter).[17a]—This paper explains the mathematical solution of a very difficult hydrodynamic problem. Particularly interesting is the fact that the result, although obtained by advanced mathematics, is given in such a convenient and simple form as to be readily usable by the average designer.

The paper and its results are of primary importance to engineers designing dams for localities where earthquakes may be expected, but it is also of use in other structural problems, as, for instance, in determining the horizontal load exerted on a building by the water in a swimming tank on an upper floor of the building.

A brief review of progress and developments in connection with the design of dams to resist earthquake oscillations, and particularly some of the specific findings in connection with the design of one dam to resist earthquakes, will be of general interest, because the next and logical question is: "What effect does it have upon the design of a dam?"

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[15] Göttingen, Germany.

[15a] Received by the Secretary January 22, 1932.

[16] Dissertation of Göttingen University, not yet published.

[17] Chf. Designing Engr., Pasadena Water Dept., Pasadena, Calif.

[17a] Received by the Secretary February 2, 1932.

Design of Pine Canyon Dam to Resist Earthquakes.—In the summer of 1928 the writer was instructed to design the Pine Canyon Dam, a concrete gravity type structure on the San Gabriel River, California, to resist the earthquakes which might reasonably be expected to occur in this locality. At that time, as far as the writer knew, no dam had ever been designed to resist earthquakes. Formulas were developed for a theoretical triangular dam designed to resist the inertia effect of the mass of the dam, and, as a result, the writer and Mr. S. B. Morris, presented a paper before the Seismological Society of America in June, 1929, entitled "Design of Gravity Dam in San Gabriel Canyon to Resist Earthquakes." [18] Then, after the World Engineering Congress in Tokyo, Japan, in October, 1929, a copy of Professor Nagabo Mononobe's paper on "Earthquake-Proof Construction of Masonry Dams," was made available to the writer, in which it was discovered that Professor Mononobe had designed and built, or had under construction, seven gravity dams designed to resist the inertia effects of the mass of the dam during earthquakes. The earliest of these dams was completed in 1927. When the proper changes in notation were made, it was found that Professor Mononobe's paper and the paper previously referred to, by the writer and Mr. Morris, contained practically the same equations although derived independently. It is seen, therefore, that Professor Mononobe's work preceded the work done by the writer and Mr. Morris by several years.

Hydrodynamic Pressure.—While investigating the forces brought upon dams during earthquake motions, attention was given to the possibility of an increased water pressure against the up-stream face of the dam as it moved toward the water. Mr. Harry O. Wood, in a discussion[19] of the report of the Committee on Arch Dam Investigation, being conducted by Engineering Foundation, stated: "There is a possible action of earthquakes on dams, dikes, etc., which appears to have been generally, if not universally, overlooked." And, still further, he summarized and stated that, if a dam moves back and forth and the water tends to remain still, being frictionless on the bottom, the water might

> "Act as a great sluggish pendulum bob, or a kind of soft-nosed battering ram of relatively great mass, which tends to remain still as the earth and the dam shift rapidly to and fro. Relatively, then, a soft-nosed, but very heavy battering ram may be caused to deliver, in rapid succession, blows or thrusts against the up-stream face of the dam."

It was suggested by the writer that if the dam moves toward the water, it would be just the same as if the water moved or flowed toward the dam, and hence the water would rise in the dam an amount equal to a velocity head, and cause an increase in pressure equal to twice the velocity head, which it was thought was somewhat analogous to the well-known problem of a jet of water impinging upon a flat surface. This was presented to two authorities on hydraulics. One answered that the foregoing viewpoint was correct, and the other thought not, and thought that the problem was a difficult hydrodynamic problem.

[18] *Bulletin*, Seismological Soc., Vol. 9, No. 3, September, 1929, p. 143.
[19] *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2609.

Pursuing the problem further, it was then noted that if a rigid surface or lid could be fixed on the water surface, the problem would be akin to water-hammer in a pipe line. However, if the lid were not there, the bottom of the reservoir would tend to act somewhat like the water-hammer in the pipe line, being restrained, whereas at the water surface there would be instant upward release of the elastic compression in the water. This suggested that the portion of water which moved with the acceleration of the dam, and which offered resistance to that movement, was bounded by some straight or curved surface, such as, for example, a 45° plane, or the parabola given by Professor Westergaard, although no definite shape or value was determined.

The aforementioned velocity-head treatment (which, incidentally, gives a negligible force in any real case), was presented in the paper by Mr. Morris and the writer.[18] However, realizing that perhaps this solution might be bettered, the following statement was made:

"The difficult hydrodynamic problem of the determination of the forces applied to the dam, owing to oscillation of the dam against the body of the water in the reservoir, deserves further investigation both in mathematical analysis and in experimentation with models on a shaking table. There are a number of high masonry dams already constructed in close proximity to active fault lines. Recording devices installed on the up-stream face of such dams placed at intervals from top to bottom of the reservoir should definitely reveal the magnitude and distribution of such stresses. This would undoubtedly afford most certain and accurate answers to the question raised by Harry O. Wood."

In the spring of 1930, a copy of the "Report of Bureau of Reclamation on Boulder Canyon Dam," which contained Professor Westergaard's original treatment of the hydrodynamic force, became available. In all essential respects this treatment was identical to that contained in his present paper.

The writer has thought for some time that the fundamental part of the hydrodynamic force ought to be susceptible of reasonably simple explanation and fairly easy derivation without the somewhat complicated mathematics used by Professor Westergaard. Numerous attempts have been made to do this, and the writer has attempted to trace the elastic actions, following the elastic waves of compression and reflected waves of both compression and dilatation, somewhat in the manner commonly used in water-hammer problems. A graphic method soon became too complicated. The writer's ideas were presented to Dr. Theodor von Kármán, and he suggested the method of images, which is only a convenient way of treating the reflections which the writer was considering. Dr. von Kármán, however, suggested that since the waves of compression radiate from a point, the intensity of the compression, between parallel planes, would vary inversely as the distance from the source, somewhat as the intensity of light varies inversely as the square of the distance from the source, and this, therefore, suggested a method of attack. Following this, Mr. C. H. Heilbron, Jr., worked out the problem along these lines.

Mathematical Check.—In a report to the Pasadena, Calif., Water Department, Eugene Kalman, M. Am. Soc. C. E., derived the same equations as the author and stated that the original Westergaard treatment agrees fully with

the general principle underlying hydraulics and that, therefore, an experimental check would not be necessary. Furthermore, he pointed out that it would be difficult because small models are influenced to a considerable extent by the point of singularity at the top of the dam. However, an experimental check has been made by Professor Lydik S. Jacobsen, on the shaking table at Stanford University, a preliminary account[20] of which shows that the experiments check the mathematics of Professor Westergaard. Professor Westergaard assumes that the dam moves in one piece as an integral part of the earth's crust. This is somewhat different from actual conditions since the natural, independent vibrations of the dam have some influence on the motion of the water. Professor Kalman pointed out the great difficulty of considering this effect and concluded that the results as presented must be accepted as a good approximation of the actual conditions.

Following Professor Kalman's report the Pasadena Water Department re-designed its Pine Canyon Dam to resist the hydrodynamic force, using Professor Westergaard's parabolic equivalent body of inertia, as well as the inertia effects upon the mass of the dam.

Effects of Earthquake Loads on the Size of a Dam.—For ease of comparison and calculation a typical triangular gravity dam section with a vertical up-stream face will be used as shown in Fig. 12.

An actual gravity dam design, straight in plan, does not differ materially from a triangular section and, hence, the general effects of any of the factors can be shown by using the simple triangular section. The principles involved in the actual design of a practical dam section with considerable top width are entirely similar to those of the ideal triangular section, except that the inter-relations cannot be expressed so simply. In addition to the symbols used by Professor Westergaard, the following nomenclature is introduced:

W = total weight of the mass of the dam.
G = specific gravity of concrete in mass of dam.
H = total static water load on face of dam.
U = total uplift pressure under dam.
V = total re-active force of foundation upon dam.
F = total force applied to dam by earthquake acceleration upon mass of dam.
D = total force applied to dam by hydrodynamic load.
h = height of dam.
B = base width.
α = ratio of horizontal earthquake acceleration to that of gravity.
r = coefficient of hydrostatic pressure to be used at heel of dam for uplift, which in this study is assumed to diminish uniformly in a straight line to zero at the down-stream toe.
j_h = horizontal acceleration of earth movement during earthquake, in feet per second per second. ($j_h = \alpha\, g$.)
j_v = vertical acceleration of earth movement during earthquake, in feet per second per second.
g = acceleration of gravity.

All the forces in Fig. 12 are shown to a true scale of masonry units, and, hence, the areas of each loading diagram represent the relative value of each

[20] *Bulletin*, Seismological Soc. of America, Vol. 21, No. 3, September, 1931, p. 204.

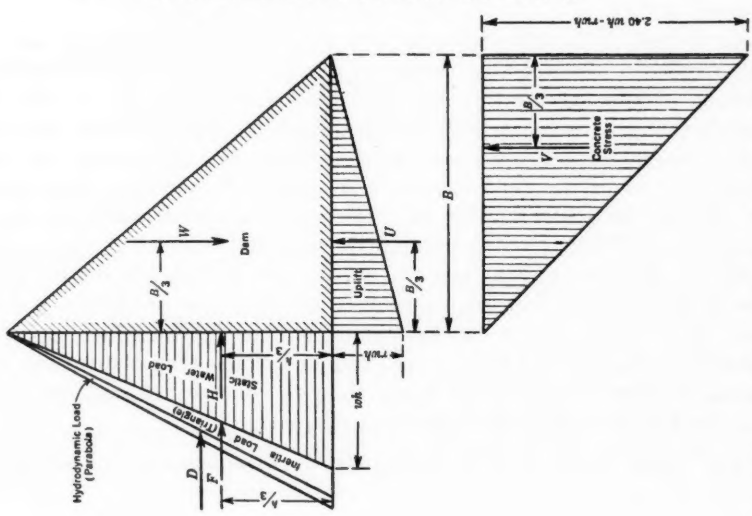

FIG. 12.—SKETCH SHOWING RELATIVE VALUES OF FORCES ON DAMS.

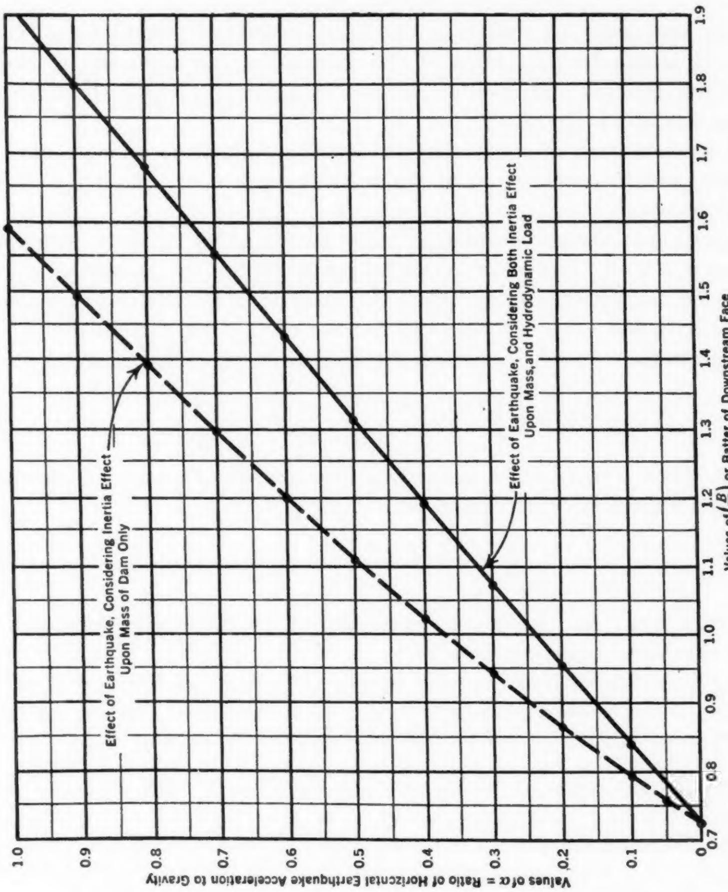

FIG. 13.—CURVES SHOWING EFFECT OF EARTHQUAKE ON SIZE OF DAM.

load. It can easily be seen that the inertia load, F, should be a triangle because it is equal to the mass of the dam times the acceleration, and the mass of the dam has a triangular section. Similarly, the hydrodynamic loading diagram will be parabolic.

For the condition of static equilibrium, the summation of moments about any point, and the summation of the vertical components of all forces, must be equal to zero.

These simple algebraic equations may be solved to derive a formula for the base width, B, in terms of the height and physical properties, for the case of the earthquake acting upon the mass of the dam only (neglecting the hydrodynamic load at first), and for the case of no earthquake.

The formula reduced, including the inertia effect only, is:

$$B = h \left[\frac{G\alpha \pm \sqrt{G^2\alpha^2 + 4(G - r)}}{2(G - r)} \right] \quad \text{.............. (82)}$$

When there is no earthquake, or when the inertia effect is not considered, $\alpha = 0$, and Equation (82) reduces to:

$$B = h \frac{1}{\sqrt{G - r}} \quad \text{...................... (83)}$$

Equation (83) has previously been derived by the writer independently without considering the inertia load.[21]

Using $G = 2.4$ (or assuming the weight of concrete as 150 lb. per cu. ft.), and $r = 0.5$ (or assuming an uplift of 50% at the heel), values for Equation (82) were calculated and plotted as Curve No. 1, on Fig. 13.

For these conditions there is only one value of $\frac{B}{h}$ for Equation (83), which is 0.725, and which checks as the point where Curve No. 1 crosses the ordinate, $\alpha = 0$. On these curves, a value of $\alpha = 0.1$ indicates an earthquake acceleration of $0.1g$.

When the parabolic hydrodynamic load is included, the relations are not so easily expressed algebraically, therefore, a number of separate arithmetical calculations were made for this case and plotted as Curve No. 2, Fig. 13. Suppose that a straight concrete gravity dam is to be designed using the foregoing unit weight of concrete, with uplift assumptions as before; and suppose that it is to be designed against a horizontal earthquake acceleration of $0.1g$. Using the curves in Fig. 13: If there is no earthquake, the downstream batter is 0.725; if the inertia effect upon the mass of the dam only, is considered, the down-stream batter is 0.793; and, if both the inertia effect and the hydrodynamic effect are considered, the down-stream batter is 0.84.

From these values it is seen that to include the inertia effect increases the material in the dam by 9.38%, and that to include both the inertia and hydrodynamic effect increases the material in the dam by 15.85% over that required for the dam designed without earthquake.

[21] "Uplift Under Dams," by C. E. Pearce, *Western Construction News*, October 25, 1928.

Actually, although Pine Canyon Dam was first designed according to the foregoing uplift assumption (50% at the heel to 0% at the toe) and by assuming an earthquake acceleration of $0.1g$ for both inertia and hydrodynamic effects, it was later designed with the same earthquake assumptions and 100% uplift at the up-stream face, diminishing uniformly to 50% at the galleries near the up-stream face, and then to 0% at the down-stream face, all considered as acting upon 100% of the area. This uplift assumption, together with the parabolic hydrodynamic load, and the proper consideration of all forces for the actual dam (not a theoretical triangle), gave the cross-section shown in Fig. 14, in which, the maximum water level used

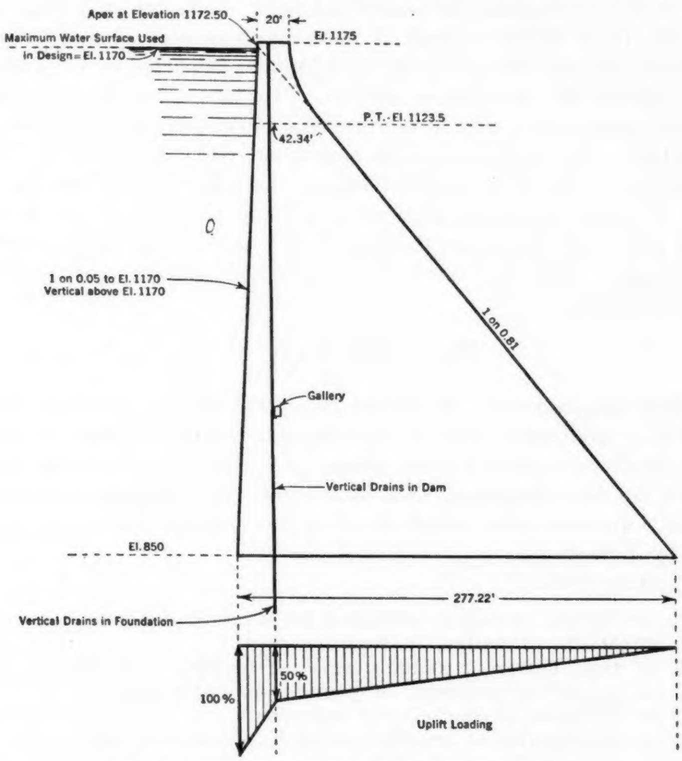

FIG. 14.—ADOPTED SECTION, PINE CANYON DAM.

in the design was 5 ft. below the roadway level. In this dam, vertical drains are 10 ft. apart in a line parallel and near the up-stream face as indicated in Fig. 14, but no allowance for such drainage was made in the design.

Vertical Earthquake Acceleration.—In any actual case of dam design, one must investigate all the possible earthquake effects, including vertical acceleration and resonance. It can be shown easily that the vertical acceleration of the earthquake has no overturning effect upon a gravity dam, because the weight of the water, and, therefore, the water pressure on the dam, is increased in the same proportion and at the same time as the mass of the

dam during the upward acceleration. In other words, an acceleration of g is acting upon both dam and water for ordinary static conditions, while an acceleration of $(g + j_v)$ is acting upon both dam and water during upward acceleration, and, hence, their moment relations are not changed. In a similar manner, a downward acceleration of the earth does not change the moment relations. The unit stresses in the dam and its foundation are increased or decreased accordingly, but there is no added tendency to overturn the structure. The vertical acceleration, however, does affect the sliding factors, and this must be investigated in each case.

Resonance.—If the natural period of vibration of the dam is the same as that of the earthquake, the elastic rebound of the stressed dam produces a force due to its kinetic energy of motion, which may come at exactly the same time as the application of the next impulse imparted by the earthquake, thereby making the two forces additive. In this manner, each additional earthquake impulse may add to the previous force, and, hence, the total force may build up. The engineer in each case must treat this matter as the situation warrants. There is ample evidence that earthquake motions do not continue as simple harmonic motions of a given period over any appreciable length of time, and, therefore, the dam would soon be out of tune again.

Timoshenko[22] gives for the natural period of vibration of a wedge the following formula:

$$T_d = 4.0952 \frac{h}{k} \sqrt{\frac{\rho}{E g}} \qquad (84)$$

Working independently, Mr. Frank Abott derived an equation similar to that of Timoshenko for a wedge by equating the potential energy of maximum deflection to the maximum kinetic energy of a vibration, assuming harmonic motion of the free vibration; and, as a result, he obtained a coefficient of 3.9738 for Equation (84), which is about 3% smaller than the coefficient obtained by Timoshenko.

In Equation (84):

T_d = natural period of vibration for dam, in seconds.
h = height of wedge, in feet.
k = total batter of wedge = ratio of breadth to height.
ρ = weight of concrete, in pounds per cubic foot.
E = modulus of elasticity of concrete.
g = acceleration of gravity = 32.2 ft. per sec. per sec.

From Equation (84), it is seen that for a dam having a constant batter, k, the natural period varies as the first power of h.

For Pine Canyon Dam, using h = 300 ft.; k = 0.86; ρ = 150 lb. per cu. ft.; and E = 360 000 000 lb. per sq. ft.; Equation (84) gives T_d = 0.16 sec.

Since the dam is 300 ft. high only at the center and is fixed and less high along the abutments, it is seen that the higher cantilevers are not only fixed at the bottom, but are also restrained, to some degree, on the sides, and that, therefore, the natural period of the dam as a whole will be even less. Conse-

[22] "Applied Elasticity," Timoshenko and Lessells, First Edition, p. 323, Equations (e) and (i).

quently, it will probably not be of much importance for the larger earthquakes having destructive value, during which the period of vibration is generally found to be much longer than 0.16 sec.

This seems to indicate that for a gravity dam of this height or less there is little cause for worry regarding resonance. For high slender arch dams, the natural period of vibration may be found to be longer, and to approach dangerously near the periods observed for destructive earthquakes. In such a case, the designer might be justified in assuming two, three, or more vibrations of the earthquake having the same period as the natural period of vibration of the dam. While a high slender arch dam tends to have longer periods, it, in turn, can take an over-stress without failure. The writer does not specifically recommend exactly the foregoing procedure for arch dams, but merely mentions it to show what might be done. As new cases arise, they must be treated as good judgment dictates, and, at the present time, any rational treatment is a forward step.

Earthquake Acceleration for Use in Design.—Since the subject of the hydrodynamic load has brought up the question of its effect on the size of a dam, it is proper, at this point, to discuss earthquake accelerations, because proper results depend upon proper premises.

In any specific case, the engineer designing a dam should investigate the records of past and probable future earthquake accelerations at the locality involved for the kind of material upon which the dam is to rest. Unfortunately, there are comparatively few data at hand that will set a definite value to acceleration for the United States, or for any particular location. However, computations have been made for such values for such major earthquakes as the California earthquake of April 18, 1906, and the Japan earthquake of September 1, 1923.[23] Study of such data is of much value in the selection of a maximum value of acceleration to be used.

Comments.—

(1) By referring to Fig. 12 on which all load areas are drawn to scale, it can be easily seen why numerous dams not designed to resist earthquakes have not failed. The sum of the inertia load plus the hydrodynamic load is small in comparison to the other loads.

(2) By increasing the cost of the dam by only about 15%, the structure may be made safe against earthquakes, and this may be a wise safeguard for a dam in a populous country where earthquakes are likely to occur.

(3) Those who have not given consideration to designing structures to withstand earthquake accelerations may be inclined at first to think that designs purporting to resist those accelerations are flighty mathematical theories. A little investigation, however, will show that the basic part of the problem is really very simple and quite easy of determination. The probable earthquake accelerations which may be expected have been measured by seismologists, and are fairly well known.

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[23] Accelerations which have occurred, or which are recommended for use or have been used by others, are discussed in the *Bulletins* of the Seismological Society, September, 1929, p. 143, and September, 1931, p. 204; in Mononobe's work referred to herein; and in two articles by Henry D. Dewell, M. Am. Soc. C. E., in *Engineering News-Record*, Vol. 100, pp. 650 and 699, entitled "Earthquake-Resistant Construction."

(4) As far as is known by the writer at this time (1932), the following dams in the North American Continent have been designed against earthquakes: The Pine Canyon Dam, on the San Gabriel River; the Hoover Dam, on the Colorado River; and the Madden Dam, in the Panama Canal Zone.

(5) It is suggested that in his closure the author give a simple physical explanation of the difference between his first case, or case of ideal water-hammer, and the true case which he has solved, showing the elastic actions that actually take place in the water.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EFFECTS OF BENDING WIRE ROPE

Discussion

By Messrs. Robert C. Strachan, and C. D. Meals

Robert C. Strachan,[9] M. Am. Soc. C. E. (by letter).[9a]—In many respects, a wire rope is similar to a solid bar. It stretches under tension and recovers when released, in a manner well known through many tests. Short pieces show regularity in measured deflections under lateral forces. Naturally, much attention has been given to the effect of passing a rope around a sheave, since one of the main purposes of a rope is to enable one to change the line of application of a force. As previously noted by the writer,[10] rope action is complicated by the presence of fiber cores, by the twisted state of the wires, and by other factors.

Analogy with a homogeneous bar is entirely lacking when a sheave is present; hence, the diversity in results based on the assumption of such an analogy is not surprising. The fact of extremely large deformations, to which this diversity must be largely attributed, will be easily perceived if values of 35 kips and 30 000 kips for b and E, respectively, are inserted in the author's Equation (3), and R is deduced. It is found thus that at the elastic limit, beyond which the formula is inapplicable, the radius of curvature of a steel specimen is approximately 430 times its depth. This would seem to show conclusively that the result of applying the common theory of flexure to a $\frac{3}{4}$-in. rope, bent around a 6-in. sheave, must be fallacious.

That there are many reasons for conflict between theory and practice is recognized by the author, who grants, in preparing to derive a rational expression for bending stress, that "the derivation * * * leads to approximate answers." One of the reasons not mentioned in the paper is the important fact that many ropes are composed of wires of different sizes. It is evident that any formula such as Equation (20), which assumes uniformity of wires, must be considerably in error for the large class of ropes in which such uniformity does not exist.

Note.—The paper by Frederick C. Carstarphen, M. Am. Soc. C. E., was published in December, 1931, *Proceedings*. This discussion is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[9] Cons. Engr., New York, N. Y.

[9a] Received by the Secretary January 5, 1932.

[10] *Civil Engineering*, November, 1930, p. 111.

The probability of error in numerical application is enhanced by the smallness of the dimensions of the elemental parts. The effect is to make minute absolute differences become large proportional differences. This may be seen in the author's application of Equation (20), where the wire diameter is taken as 0.054 in. So small a difference as 0.001 in this dimension leads to a difference of 6% in the calculated value of P.

The presence of error due to the exclusion of the effect of direct tension in deriving Equation (20) is recognized by the author, but it is altogether probable that this error is of greater importance than his somewhat casual reference to it would indicate.

Taking into consideration the complex mechanics of combined tension and torsion in a compound wire helix, the large deformations, the small dimensions of parts, the internal conditions not discernible by visual inspection, but in fact affecting core resistance, and the variety of materials, as well as the different wire sizes in one rope, it appears to the writer impracticable to derive a satisfactory rational formula for bending stress. In the last analysis, reliance must be placed on the "life-saver" coefficient derived by experiment.

The comparatively simple Equation (21) would seem preferable in form to Equation (20); but a better line of approach to the problem is that taken by the National Bureau of Standards, in making and recording tests which show how much loss in strength is caused by passing a rope around a sheave. It is to be regretted that the published results do not cover a more extended range of both rope types and sheave sizes. In the writer's opinion, an exhaustive series of tests such as these, would be of far greater benefit than any rational formula for bending stress, however plausible its bases and beautiful its derivation.

C. D. MEALS,[11] ASSOC. M. AM. SOC. C. E. (by letter).[11a]—The wire of the ancients was produced solely by hammering and not by the process of "drawing" as practised to-day. The "drawing" of wire was mentioned in an old Latin manuscript by Theophilus, written during the Eighth or Ninth Centuries. Undoubtedly, the art was known and practised before that date. Authentic records are available to show that wire drawing, on a commercial scale, was practised in France, in 1270, in Germany, in 1350, in England, in 1465, and in America, in 1775. All these wire ropes were made on a "rope-walk," following the practice used in the making of hemp ropes.

It was a simple beginning of a business of great importance; there are sixteen wire-rope manufacturers in America to-day (1932), capable of producing more than 150 000 tons of wire rope annually, excluding ordinary strand as used by the public utility companies. Of this number, only three make their wire from the open-hearth furnaces to the finished product; most of the others draw the wire from rods purchased from the steel manufacturers.

It is doubtful whether Albert of Clausthal made lang lay ropes, since John Lang, of England, patented this lay in 1879; although in Germany it is

[11] Wire Rope Engr., The B. Greening Wire Co., Ltd., Hamilton, Ont., Canada.

[11a] Received by the Secretary February 9, 1932.

called Albert Lay. It is easy to visualize the possibility that hemp rope manufacturing practice influenced the early wire-rope making, and hemp ropes have always been made "regular" lay.

The development of the various constructions and lays of ropes started in 1875. The non-spinning and flattened strand ropes and locked-coil cables were first made and patented in England, and the British manufacturers have developed these ropes to a high degree of efficiency, while American manufacturers have lagged decidedly. American manufacturers, however, have developed the round strand ropes to a much higher degree of efficiency than those made in other countries.

Prior to 1854, all wire rope was made of iron wire, a grade that is relatively soft and easy to handle. The first of the higher grades of steel wire, as used so extensively to-day was first produced in England in 1854; it was a tempered or "patented" wire and represented a great advance in the trade or art.

Mechanical Development of Machines.—The Stone stranding machine is different in construction than is implied from the description given of it, following its mention in the paper. It was found unsuited for high-class wire-rope making and is not used to-day. Instead the "rod and disk" strander, as described by Mr. Carstarphen, and the tubular strander, are the high-speed horizontal stranding machines now in use. The tubular strander is the more rugged and costs less to maintain than the "rod and disk" type machine, but it has the disadvantage of being more difficult to load with bobbins, which slows up production.

The stranders described herein may have from seven to thirty-seven bobbins arranged in one machine; the latter makes an extremely long unit, and one in which a tubular machine is practically a necessity; or the machine may be split in two units and geared to drive together at the same speed, or at varying speeds.

The tandem horizontal stranders as described by Mr. Carstarphen are of the old English type, which are being supplanted to-day by the high-speed stranders described in the foregoing paragraphs. It is merely a question of production; the high-speed stranders will produce two to three times as much strand per day as the tandem horizontal stranders, at no sacrifice in quality.

Some of the larger companies have vertical stranders in operation, particularly for making the more flexible rope strands. All these machines have a planetary motion imparted to the bobbins, and many are geared to give a back or forward turning motion to the bobbins for making strands for lang lay ropes. It is possible to provide the bobbins in the tubular high-speed stranders with a back-turning motion by a set of ring and internal gearing.

At least six of the American firms have stranding equipment capable of laying 37 wires together in one operation and several have stranders that will lay 41, 43, and 46 wires. Some of these large vertical stranders will make up 20 tons of strand at one setting of the machine.

The "closing" or "laying" machines, in which six or eight strands are laid around the hemp center to form the finished rope, are either horizontal or vertical, with a planetary motion imparted to the spools. They are often

geared to give a back turn to the spools for making lang lay ropes and, in addition, may be equipped with a straightening device to neutralize the liveliness of a lang lay rope still further.

Several of the American firms have large vertical closers, capable of making 60 tons of rope at one setting of the machine—a machine that will make 20 000 ft. of 2-in., or 8 000 ft. of 3-in., steel wire rope. It is possible, however, to make 30 tons of rope in one continuous length on a closer of 15-ton capacity.

The compound strander and closer mentioned by Mr. Carstarphen is known as the "Canadian" machine, as it was first built in Canada. It is interesting as an exhibition model; but it was not practical for several reasons, such as, (1) reduced production, and (2) an inferior and cranky rope.

Characteristics of Wire Ropes.—Strands made up "as a series of single operations," as described by Mr. Carstarphen—as, for example, a 37-wire strand made up by laying 18 wires over 12 over 6 over 1 wire—are not offered to the trade by the better rope makers, as it is a well known and established fact that a rope strand of the "one-operation" class is far superior to a rope strand of the "multiple-operation" class. The former has each wire in the strand interlocked with each adjacent wire; the latter has wire crossing wire within the strand, resulting in internal friction and a decidedly inferior rope.

Impregnated hemp centers were an improvement over the old method of running a center through a bath of hot lubricant which did not penetrate, because a hemp center is, next to lariat and yacht rope, the hardest hemp rope made. Impregnation of the centers has some detrimental points; one is that in drying out the centers in a vacuum tank, the fibers lose their resilience to a certain extent and the original diameter of the center is not maintained. Consequently, the progressive makers are now using pre-lubricated centers in which every fiber is given a spraying of hot lubricant before being spun into the yarns. Hemp centers for wire ropes should be uniform in both diameter and hardness. The quality of fiber may well be submerged, within reasonable limits, to these two properties.

Consideration of a wire rope as a machine, having many moving parts and wearing surfaces, should force the realization that it must be lubricated if satisfactory rope service is to result. Proper lubrication retards corrosion and rotting of the hemp center; it decreases external wear of rope and equipment and reduces friction between the component parts of the rope. Varying rope service requires different kinds of lubrication and, primarily, the lubricant should be "built" into the rope during its manufacture, both at the stranding and closing machines, with exterior applications, as required, when the rope is in service.

Admittedly, sheave and drum equipment are generally too small for satisfactory rope service; power and space requirements set a limit to their sizes, but it must be appreciated that tight grooves in sheaves, that wedge or pinch the rope, prevent the unrestricted wire and strand movement that is so necessary to least resistance to bending. Consequently, such grooves subject the rope to greatly increased bending stresses—causing premature breaking of

the wires and preventing a new rope from rotating freely—and increase the wear on the wires of the rope.

A new wire rope, operating under varying stresses, will elongate, and this lengthening of the lay is equalized by an untwisting or rotating action of the rope. In a regular lay rope, the wires in the strands are laid in the opposite direction to that of the strands in the rope. Consequently, any untwisting will tighten the wires in the strand and aid in the resistance of the rope to untwisting.

In a lang lay rope, the wires and strands are laid up in the same direction and, consequently, an untwisting action will cause an opening out or "bird-caging" of the individual wires of the strand, because every two and one-half rotations of the rope result in a full rotation of the individual strands. This causes the wires of the strands to "bird-cage," and the strands in the rope will also "bird-cage" or arch away from the hemp center.

An important fact is inferred in the foregoing—never use a lang lay rope where the untwisting action cannot be controlled or limited. Consequently, a lang lay rope should not be used where its ends are not permanently fastened; where a swivel is used; nor where grooves in the sheaves are tight. There is some limitation to this in the sense that a coarse and stiff rope,

TABLE 5.—STANDARD CHARACTERISTICS OF WIRE ROPE

Rope	Outer Wires in Strands		Regular Lay Ropes		Lang Lay Ropes	
	Number	Diameter, in inches	Strand lay	Rope lay	Strand lay	Rope lay
6×7	6	0.106	2½ *d*	6¾ *d*	2¾ *d*	7½ *d*
6×19 Seale	9	0.080	2½ *d*	6¼ *d*	2¾ *d*	7 *d*
6×19 filler wire Seale	12	0.064	2⅝ *d*	6¼ *d*	3 *d*	7 *d*
6×19 Warrington	12	{0.054 0.072}	2⅝ *d*	6¼ *d*	....	....
6×37	18	0.046	2⅞ *d*	6¼ *d*	3⅛ *d*	7 *d*
8×19	12	0.052	2 *d*	6¼ *d*	....	....
6×8 flattened strand	7	0.108	....	....	2¾ *d*	7½ *d*
6×25 flattened strand	12	0.073	....	....	2¾ *d*	7 *d*

as a 6 × 7 or a 6 × 8 flattened-strand lang lay rope, will not "bird-cage" as easily as the more flexible 6 × 19 ropes, because the large wires of the stiff ropes offer considerable resistance to "bird-caging."

Strand and rope lays are not usually given in terms of the wire diameters, as indicated in Table 1, but in terms of the rope diameter, and the outer wires of the strands are fixed by allowance for the angle of twist of the strand and a proper clearance between adjoining wires in the same layer. Accordingly, Table 1 may well be changed to read as shown in Table 5 for a 1-in., or base-size, rope.

These values correspond quite closely with those in Table 1, but it must be understood that each manufacturer has his own ideas regarding strand and rope lays. Table 5, however, represents average American practice for high-class operating ropes. For galvanized guy ropes and other stationary ropes as used in suspension bridges, etc., the rope lays may average **8*d* to 9*d* lays.**

The unit tensile strengths of wire for any grade vary with the wire diameter, as indicated for plow-steel quality in Table 6 (Item Nos. 1 to 4), Mild cast-steel wire is now termed "traction steel" since it is used for elevator rope service and the super-plow steel ropes are known as "improved plow steel," with the makers' trade designations for this quality of wire.

Torsion requirements for wires are usually given in terms of twists per length of 100 d', even if the tests are made in a gauge length of 6 in. or 8 in. They vary as indicated in Table 6 (Items Nos. 5 to 19). Small wires will show higher torsions than large ones of the same quality, although this fact is not considered in most commercial specifications.

American mills are very reluctant to furnish galvanized wire to a torsion requirement. In fact, some will not accept orders on this basis; but the British and German mills will do so, and the British and Canadian Engineering Standards Associations give torsion values as shown in Table 6 (Items Nos. 11 to 15). Some recent vertical lift-bridge specifications for galvanized improved plow-steel wire are as listed in Table 6 (Items Nos. 16 to 19).

Elongations of rope wire will vary with the diameter of the wire, the process of "patenting" (as "air-patented" or "lead-patented"), and the quality of the wire. Values in Table 6 (Items Nos. 20 to 24) are typical of minimum elongations at breaking loads.

There is still another physical property of rope wire that may be of interest, namely, the hardness. Item No. 25 of Table 6 indicates the relative abrasive and wearing resistance of the various grades of wire in terms of the Brinell hardness number (B H N). It will be found that a fairly close relation exists between the tensile strength and the B H N from the formula given by Abbott,[12] in units of 1 000 lb., as follows,

$$S = 0.73\ [\mathrm{B\ H\ N}] - 28 \qquad (33)$$

The chemical properties of the various grades of rope wire may be of interest; while each manufacturer has his own ideas, the values in Table 6 (Items Nos. 26 to 33) are typical specifications.

Acid or Basic Open-Hearth Steel?—A few years ago the writer conducted a series of bending tests on wire ropes of the various grades and constructions, and it was shown that a good basic steel was as satisfactory as a good acid steel, but the writer favors the acid steel because it is more likely to be consistently good than the basic steel.

The question is occasionally raised, "Why do not the wire rope manufacturers use alloy steel for wire ropes?" Some research work has been done on nickel, vanadium, molybdenum, and stainless (chromium and nickel) steel wire ropes and some of them have shown considerable merit; but the price of these alloy steels is relatively so prohibitive that they are not commercially practical for wire ropes.

Wire-rope strengths were revised and generally lowered in 1929; on some grades, sizes, and constructions of ropes it was a necessity. In a general sense, it was a move in the right direction since recent research has indicated

[12] "Strength and Hardness of Steels", *Proceedings*, Am. Soc. for Testing Materials, 1915, p. 42.

TABLE 6.—PHYSICAL PROPERTIES OF WIRE ROPE

Item No.	Identity	Iron	Traction steel	Cast steel	Extra cast steel	Plow steel	Improved plow steel
	TENSILE STRENGTHS, IN KIPS PER SQUARE INCH						
	Diameters of Wire, in Inches:						
1	0.010 to 0.083	....	....	....	....	230 to 255	....
2	0.084 to 0.132	....	....	....	....	225 to 250	....
3	0.133 to 0.170	....	....	....	....	220 to 245	....
4	0.177 to 0.204	....	....	....	....	212 to 235	....
	TORSION REQUIREMENTS FOR STEEL WIRE, IN TWISTS PER 100 d'						
5	Typical Rope Manufacturing Practice†	80 to 120	33 to 38	30 to 35	27 to 32	25 to 30	25 to 29
6	American Petroleum Institute*	....	....	32.5	30	27.5	....
7	United States Government*	....	....	33	30	28	25
	Engineering Standards Associations:						
8	British†	....	....	34	32	30	28
9	British*	....	....	26	24	23	21
10	Canadian†	....	....	25	25	25	
	TORSION REQUIREMENTS FOR GALVANIZED STEEL WIRE, IN TWISTS FOR 100 d'						
	Engineering Standards Associations:						
11	British†; 0.104 in. and less	....	....	28	23	15	10
12	British*; 0.105 in. to 0.144 in.	....	....	23	19	11	7
13	British*; 0.104 in. and less	....	....	21	17	11	8
14	British*; 0.105 in to 0.144 in.	....	....	17	14	8	5
15	Canadian†	....	....	15	15	15	....
	Vertical Lift-Bridge Specifications:						
16	0.027 to 0.051 in.	....	....	....	....	....	20
17	0.052 to 0.100 in.	....	....	....	....	....	18
18	0.101 to 0.125 in.	....	....	....	....	....	17
19	0.126 to 0.190 in.	....	....	....	....	....	16
	PERCENTAGE ELONGATIONS OF STEEL WIRES IN 10-INCH LENGTHS						
	Diameters, in Inches:						
20	0.010 to 0.026	....	....	1.5	....	1.25	1.0
21	0.027 to 0.051			2.0		1.5	1.5
22	0.052 to 0.075			2.5		2.0	1.75
23	0.076 and more			3.0		2.0	2.0
24	Galvanized wire, 0.010 in. and more	....	....	5.0	....	4.0	....
	HARDNESS TEST						
25	Brinell Hardness Number	150	250	340	370	390	420
	CHEMICAL PROPERTIES (PERCENTAGE CONTENT)						
26	Carbon	0.06 to 0.12	0.30 to 0.40	0.45 to 0.55	0.55 to 0.65	0.63 to 0.73	0.70 to 0.80
27	Manganese	0.15 to 0.30	0.40 to 0.60	0.40 to 0.60	0.40 to 0.60	0.40 to 0.60	0.30 to 0.50
28	Silicon	0.10 (maximum)	0.10 to 0.20	0.10 to 0.20	0.10 to 0.20	0.10 to 0.20	0.10 to 0.20
29	Phosphorus (maximum values)	0.04	0.04	0.04	0.04	0.04	0.04
30	Sulfur (maximum values)	0.04	0.04	0.04	0.04	0.04	0.04
31	Chromium (maximum values)	0.04	0.04	0.04	0.04	0.04	0.04
32	Nickel (maximum values)	0.04	0.04	0.04	0.04	0.04	0.04
33	Copper (maximum values)	0.10	0.10	0.10	0.10	0.10	0.10

* Tests of wires from finished ropes.

† Tests of wires from coils before being stranded for ropes.

that wires of lower strength will withstand bending over sheaves better than wires of excessively high strength. Therefore, while ropes can be furnished, in most cases, to the old lists, the new strengths permit of the usage of a better steel for operating ropes.

Foreign ropes are listed at higher strengths than American ropes; but practices abroad differ in that longer lays of ropes prevail. It should be kept in mind that sheaves and drums on equipment abroad are of more liberal proportions than those in the United States. Consequently, each country has developed rope practices to suit its own particular requirements, and the higher strengths of foreign ropes need not be considered as a "challenge" to American manufacturers.

Admittedly, wire ropes made to foreign standards are unsuited for American equipment, but if made to American standards (and the writer has had this done) there is very little, if any, choice between the merits of American and of Canadian, English, and German wire ropes.

Properties of Wire Rope; Analysis of Strength.—For the usual wire-rope problems, detailed calculation of strength is scarcely required, as the approximate breaking strengths as given in the manufacturers' lists will serve most purposes just as the properties of structural shapes given in the steel handbooks serve the structural engineer. However, it may be desirable to make calculations for strengths for special ropes, such as suspender ropes for suspension bridges, vertical lift-bridge ropes, etc.

TABLE 7.—Efficiencies of Wire Ropes

Rope	Equation (34)		Metallic area in 1-in. rope (base size), in square inches	Test Results		British Engineering Standard Association long lay rope
	Regular lay	Lang lay		Regular lay	Lang lay	
6 × 7	86.5%	88.8%	0.3893	87%	90%	92.5%
6 × 19 Seale	84.8%	87.6%	0.4000	85%	88%	87.5%
6 × 19 filler wire Seale	85.5%	88.5%	0.4063	85%	88%	87.5%
6 × 19 Warrington	85.8%	......	0.4034	85%	....	87.5%
6 × 37	84%	86.9%	0.3803	82%	85%	82.5%
8 × 19	83.5%	......	0.3658	82%	....	......
6 × 8 flat; strand	......	88%	0.4766	....	....	92.5%
6 × 25 flat; strand	......	85.4%	0.4782	....	....	87.5%

A more accurate method of calculating the strengths of wire ropes than that given by the author (see "Strength of a 6 × 19 Rope"), given by:

$$S = N \cos \beta \left(\sum_{1}^{n} S' \cos \alpha^3\right) \quad \text{.................. (34)}$$

which for a 6 × 19 rope becomes:

$$S = 6 \cos \beta \left(S'_1 + 6\, S'_2 \cos \alpha^3_2 + 12\, S'_3 \cos \alpha^3_3\right) \quad \text{........ (35)}$$

Table 7 contains efficiencies of wire ropes listed in Table 5.

For the two examples given by Mr. Carstarphen, the efficiencies as given by Equation (34), are as follows:

	Equation (34)	Carstarphen
1-in. 6 × 7 cast-steel rope........	86.6%	83.7%
1-in. 6 × 19 cast-steel rope........	83.4%	78.5 and 79.9%

For improved plow-steel and for iron ropes, the efficiencies as given by Equation (34) should be reduced by 5 per cent. This value is in conformance with test results and may be explained by the extremely high pressures and wire nicking in the improved plow-steel ropes and the wire nicking in the relatively soft iron ropes. Therefore, for 6 × 19 rope, 85.5% × 95% = 81.2%, etc. A check of Equation (34) is offered by the strengths of ropes used on some well-known bridges, as noted in Table 8.

TABLE 8.—STRENGTHS OF SPECIAL ROPES

Rope	Use	Area, in square inches	By Equation (34)		Average of Tests	
			Strength, in kips	Efficiency	Strength, in kips	Efficiency
1¾-in., 6 × 19....	Counterweight ropes, NewarkBay vertical lift bridge Central Railroad of New Jersey.	1.223	255	88.5%	263	91%
2⅛-in., 6 × 19....		1.729	363	89.7%	367	90.6%
1⅞-in., 6 × 19, with 7 × 7 IWRC	Suspender ropes, Ambassador Bridge, Detroit, Mich.	1.668	308	86%	303	84.5%
2¼-in., 6 × 37, with 7 × 7 IWRC	Suspender ropes, Philadelphia-Camden Bridge.	2.207	439	90.3%	436	89.2%

In calculating strengths of ropes with independent wire rope centers (IWRC) the strength of the IWRC is only 80% effective in the rope on account of the heavy pressures on it by the enveloping 6-rope strands. Consequently, a 7 × 7 IWRC will be only 0.865 × 0.80, or 69% efficient

Equation (34) is also of value in calculating the strength of ropes with broken or worn outer wires. In the case of broken wires, it is of value when the breaks are distributed over the various strands and not concentrated in one strand; the latter case leads to an unbalanced distribution of stress which cannot be calculated accurately.

A check of the foregoing is the tests[13] reported by the U. S. Bureau of Mines. In one test, three large and three small outer wires of a 6 × 19 Warrington rope were cut in each of the six strands. Considering metallic area, 68.8% of the rope strength remained, and by test the efficiency was 74.5% and by Equation (34), 72.6 per cent.

Incidentally, it is of interest to note that American manufacturers do not include the strength of the solid triangular wires in the cores of the strands of their flattened strand ropes; the English, Canadian, and German firms do. This fact was disclosed by Equation (34), and it should be kept in mind if a comparison of strengths is made. It is known that these solid triangular wires break up in quite short lengths when such ropes are operated over small sheaves and drums; consequently, it is advisable to neglect the strength of these shaped wires in the strength of flattened strand ropes.

Bending Stress in Wire Rope.—The determination of the bending stresses is in the "battle zone" of wire-rope engineering and no two writers agree on the subject. The writer has stated his views in two previous discussions[14]

[13] "Rules and Regulations for Metal Mines", *Bulletin No. 75,* U. S. Bureau of Mines, 1915.

[14] "Aerial Tramways", by F. C. Carstarphen, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 92 (1928), p. 966; and "Heavy Duty Wire Ropes and Sheaves", by B. R. Leffler, M. Am. Soc. C. E., *Civil Engineering,* Vol. 1, No. 2, November, 1930, p. 110.

and still considers that such stresses are not susceptible to mathematical deductions with any reasonable degree of accuracy.

Consider the results of bending tests made of $\frac{5}{8}$-in. ropes on a fatigue bending test machine, in which the ropes were reeved on three sheaves placed closely together. The rope was subjected to a reverse bending operation over and under the three 12-in. tread sheaves, under constant tensions of 1 500 and 2 000 lb. Five different series of tests will be given; after which the reader may decide that bending stresses are an uncertain quantity after all.

Example 1.—In two comparable specimens of 6 × 37 plow-steel rope, $\frac{5}{8}$ in. in diameter, the diameters (in inches), were as follows:

	Rope A	Rope B
One core wire	0.036	0.033
Six inner wires	0.033	0.030
Fifteen inner wires	0.025	0.023
Fifteen outer wires	0.033	0.033

It will be noted that the strand and rope lays were identical as well as the diameter of the hemp center. The outer wires were also identical, and yet the result of the bending tests (in number of bends) was as follows:

Broken Wires	Rope A	Rope B
First outer wire	87 000	30 000
Forty other wires per foot	150 000	67 500
Rope pulled apart	425 000	210 000

This was purely a case of proper and improper strand design, most certainly not susceptible to mathematical analysis.

Example 2.—Two samples of two types of $\frac{5}{8}$-in., traction steel elevator ropes, under a tension of 1 500 lb., were tested with the following results (in number of bends):

	First Piece: First wire	First Piece: Rope failure	Second Piece: First wire	Second Piece: Rope failure
$\frac{5}{8}$-in., 6 × 27 Seale.......	40 800	244 300	48 000	341 800
$\frac{5}{8}$-in., 6 × 19 Warrington.	40 800	121 600	51 000	170 000

In each case the first and second pieces were cut from the same reel and the tests show a difference of approximately 40% at rope failure.

Example 3.—Samples of $\frac{5}{8}$-in., 6 × 19 Seale traction steel elevator ropes, under a tension of 1 500 lb., were subjected to both reverse bending (three sheaves) and continuous bending (over one sheave in one plane only), with the following results (in number of bends):

	Forty outer wires broken per foot
Reverse bending	102 000
Continuous bending	270 000

The effect is scarcely susceptible to mathematical analysis.

Example 4.—The effect of strand construction for $\frac{5}{8}$-in., 6 × 19 and 6 × 37 plow-steel ropes, under 2 000 lb. tension, is shown in Table 9. The outer wires were identical in each type of wire and the difference in strand construction is seen to be scarcely susceptible to mathematical analysis in comparison to test results.

The writer saw a striking demonstration of Example 4 on a standard type of steam shovel, equipped with two $2\frac{1}{8}$-in., 6 × 37 lang lay (IWRC) hoist lines. Two ropes made of "three-operation" strand gave only 12 and 13 days' service, respectively; while the 6 × 37 filler wire ropes were giving an average of 33 days' service.

TABLE 9.—RELATIVE BENDING LIFE OF ROPES OF DIFFERENT STRAND CONSTRUCTIONS

Rope	Strand Details		Bends of Rope	
	Construction	Type strand (operations)	Forty wires broken per foot	Rope failure
6 × 19 regular	12/6/1	Two	34 000	59 000
6 × 19 filler wire Seale	12/6/6/1	One	105 600	158 400
6 × 37 ordinary	18/12/6/1	Three	82 500	156 700
6 × 37 regular	18/19	Two	104 000	212 000
6 × 37 filler wire	18/9/9/1	One	121 000	312 000

Example 5.—Lubrication, or lack of it, greatly influences the bending life of wire ropes, as Biggart's tests made in 1890 for the Forth Bridge on $\frac{9}{16}$-in., 6 × 19, plow-steel ropes show in the following (number of bends):

	10-in. sheave	24-in. sheave
Dry rope	16 000	74 000
Lubricated rope	38 700	386 000

The foregoing tests (covering strand construction, Examples 1 and 4; reverse bending, Example 3; lubrication, Example 5; and manufacturing variation, Example 2) clearly indicate that bending stresses are not susceptible to mathematical analysis. Nevertheless, such stresses must be considered, even if empirically, and the writer prefers the formula:

$$b = \frac{E_r d'}{D} \qquad (36)$$

in which, values of E_r, the modulus of elasticity of the rope for various rope constructions, are given in the writer's discussions[14] of the two papers, previously mentioned. Equation (34) allows for an increase in E_r values for ropes that have been in service some time, which is an important consideration not fully allowed for in other bending stress formulas.

As to the factor of safety on load, acceleration, and bending stresses, experience is the only safe guide, and R. C. Strachan, M. Am. Soc. C. E., covers[15] this point in his discussion of Mr. Leffler's paper, in which he stated that "the choice of a value for k [a modifier of E] should be governed by the conditions of the particular job and the known results in work of a similar character."

Incidentally, the variables of wire ropes as indicated by tests noted in Examples 1 to 5, will explain the difference in service of the $\frac{5}{8}$-in. elevator ropes in a Cincinnati, Ohio, building, as given in Table 4. Similar varia-

[15] *Civil Engineering*, Vol. 1, No. 2, November, 1930, p. 112.

tions in elevator rope service were noted[16] by Mr. C. T. Coley and such variations are typical of many wire-rope installations, particularly in elevator service, because it is doubtful whether equal tensions are maintained in a set of six hoisting ropes on an elevator in its travel up and down the shaft, unless some type of equalizer is used on the ropes.

It was thought that Equation (20) would be applicable to wire ropes under static loading, such as suspender ropes on suspension bridges. As the limitations noted by Examples 1, 2, and 4, would not be applicable to such ropes, a check was made with Skillman's tests[17] of ropes over sheaves and of tests[18] of the $2\frac{1}{4}$-in., 6 × 37, suspender ropes of the Philadelphia-Camden Suspension Bridge, and these results, together with tests of the $\frac{3}{4}$-in. ropes noted by Mr. Carstarphen, are arranged in Table 10.

TABLE 10.—TESTS OF ROPES OVER SHEAVES

Rope	Diameter of sheave, in inches	$\frac{D}{d}$	Breaking Strengths, in Kips: Straight rope	Breaking Strengths, in Kips: Over sheave	Efficiency: Tests	Efficiency: Equation (20)
Skillman:						
¾-in, plow steel, 6 × 19	10	13.3	49.7	39.8	80.3%	97.3%
1¼-in, 6 × 19, plow steel	10	8	130.7	99.1	75.8%	95.4%
1¼-in., 6 × 19, plow steel	18	14.4	130.7	110.6	84.7%	97.3%
Carstarphen:						
¾-in., 6 × 19, cast steel	6	8	35.0	32.0	91.5%	91%
2¼-in., 6 × 37, with IWRC	32	14.2	436.5	388.4	89%	98.5%

Skillman's tests covered a series of 6 × 19, plow-steel ropes, $\frac{5}{8}$ in., $\frac{3}{4}$ in., $\frac{7}{8}$ in., 1 in., and $1\frac{1}{4}$ in. in diameter, over sheaves, 10 in., 14 in., and 18 in. in diameter, and the results obtained for the various ratios of $\frac{D}{d}$ were quite consistent. They check quite closely with the efficiency given by the $2\frac{1}{4}$-in., suspender ropes when the difference in construction and flexibility of the latter are taken into consideration, and it is rather difficult to reconcile the great difference in efficiencies between these tests and those reported by Mr. Carstarphen for $\frac{3}{4}$-in. ropes.

It is obvious from Table 10 that Equation (20) is not satisfactory to use for suspender ropes, slung over cable bands on the main cables of suspension bridges, and for similar reeving of ropes over sheaves. There is a need for a solution of this problem, and the writer has been using the formula given by Mr. Leffler,[19] which is modified by the efficiency factor, ε, as follows:

$$S = A \left(t - \frac{E_r d'}{D + d} \right) \varepsilon \qquad (37)$$

[16] Third Conference on Wire Rope Research, under the Auspices of American Engineering Council and American Society of Mechanical Engineeers, September 13, 1929.

[17] "Some Tests of Steel Wire Ropes on Sheaves", by E. Skillman, *Paper No. 229*, U. S. Bureau of Standards, 1923.

[18] Final Report of the Board of Engineers to the Delaware River Joint Commission of Pennsylvania and New Jersey, 1927, p. 118.

[19] "Heavy Duty Wire Ropes and Sheaves", by B. F. Leffler, M. Am. Soc. C. E., *Civil Engineering*, Vol. 1, No. 2, November, 1930, Equation (4).

in which, t is the ultimate tensile strength of the wire, in pounds per square inch, and ε is the efficiency of rope in Equation (34).

In the discussion of Mr. Leffler's paper, the writer criticized the use of Equation (37) for bending stresses in operating ropes; but for a non-operating or stationary rope, it gives an approximation that is on the safe side, which results in a lower efficiency than that observed by tests. As examples, for the $2\frac{1}{4}$-in. rope in Table 10, the value of S becomes 385.1 kips, as compared to test results of 388.4; for the $1\frac{1}{4}$-in. rope on the 18-in. sheave, it is 104.8 kips, compared to 110.6 by test; and for the $1\frac{1}{4}$-in. rope on the 10-in. sheave, it is 91.5 kips, compared to 99.1 by test. For the lower ratios of $\frac{D}{d}$, the error is a maximum.

The New Target, A Service Factor Formula, Equation (21).—Much that has been noted in the foregoing discussion of bending stresses, will indicate that so little, really definite, is known about the mechanics of wire rope, that engineers are not ready yet for the advance suggested by Mr. Carstarphen. At best, all that could hope to be done with such a formula, would be to set up different constants, factors, and exponents in Equation (21) for varying services, as elevators, mines, shovels, tramways, etc., as well as for the many different constructions, lays, and grades of ropes. Then, the "yardstick" would be difficult of application because no two manufacturers make their ropes alike, even for, presumably, the same rope.

Compared to many other engineering materials and products, the knowledge of wire rope is quite limited and, unfortunately, the manufacturers are very reluctant to join in any outside research work to improve the art, such as that sponsored by the Wire Rope Research Board of Engineering Foundation and the American Society of Mechanical Engineers, the last report of which appeared in 1929. The British and German engineering societies have done considerably more research work on wire rope and are being more actively supported by the manufacturers. They have published some excellent reports of their results as accomplished to date (1932), but most of the work is still ahead of them before anything definite on the subject is known.

It must not be considered that the American manufacturers have lagged in research work, such a monumental work as the George Washington Bridge attests to this fact and at least five of the leading manufacturers have research and adequate testing facilities, not only for finished ropes but also for individual wires. The results are kept confidential, however, and until either the National Bureau of Standards or Engineering Foundation equips a laboratory with suitable machines and competent research workers to carry on investigations on wire and wire rope, independent of any aid or encouragement from the wire-rope manufacturers, will anything of definite value be known about the subject.

Had the works of predecessors and contemporaries in the various engineering sciences, such as those of Galileo, Newton, Kelvin, Rankine, St. Venant, Winkler, Whipple, Einstein, etc., been withheld from publication, engineering sciences would have been greatly retarded. Their works have served a two-

fold purpose: To create an acquaintance with their thoughts and, equally important, to encourage others to think, because the real value of a book is in what it "pulls" out of one rather than what one gets out of it.

Wire-rope engineering owes a debt of gratitude to the writings of Roebling, Hrabak, Vaughan, Epton, Hewitt, Bucknall Smith, Chapman, Adamson, Howe, Sunderland, Carstarphen, and many others. It is to be hoped that in the near future much more will be added to the present meager knowledge of the subject by competent men who are engaged in the industry.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

WIND-BRACING CONNECTION EFFICIENCY

Discussion

By Messrs. David Cushman Coyle, William R. Osgood, O. G. Julian, Harold S. Richmond, and Robins Fleming

David Cushman Coyle,[5] M. Am. Soc. C. E. (by letter).[5a]—The author is quite correct in stating that it is sometimes unsafe to disregard the dead load negative moments which are to be added to the wind moments usually figured on the beam connection. His analysis in detail is, however, open to some qualification.

Consider the connection shown in Fig. 3 (*a*), regarding the possibility of loosening the rivets. Assume two $\frac{3}{4}$-in. rivets for which A equals 1.6 in., t is 1.4 in., and $C = 1\frac{1}{2}$ in. Let the total pull equal the yield-point strength of the two rivets, 38 320 lb. Then, by Equation (2*a*):

$$2R = 38\,320 \times \frac{(0.67 + 1)}{1} = 64\,000$$

As this stress is far above the yield point, the rivets will yield until they are as in Fig. 3 (*b*), when the rivet stress will have become $2R = 2S = 38\,320$ lb. Let $b = 3$ in. so that the flange is just strong enough to hold this load at its own yield point. Then, at the rivet, C being $1\frac{1}{2}$ in.:

$$\frac{dy}{dx} = -\frac{WA^2}{2IE} = -\frac{19\,160 \times 1.4^2}{60\,000\,000 \times 0.69} = 0.00091 \times 1.5 \text{ in.} = 0.0014 \text{ in.}$$

that is, at the point when R becomes equal to S the total elongation of the rivet is 0.0014 in. Meanwhile, from Equation (4), if $g = 2$ in.:

$$\Delta_R = \frac{(19\,160 - 13\,410) \times 2 \text{ in.}}{30\,000\,000 \times 0.44} = 0.0009 \text{ in.}$$

which is the elastic stretch.

On removal of the load, the rivet contracts 0.0014 in., reducing the tension in the rivet; thus:

$$\frac{-0.0014 \times 30\,000\,000 \times 0.44}{2} = -9\,200 \text{ lb.}$$

Note.—The paper by U. T. Berg, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[5] Cons. Engr., New York, N. Y.

[5a] Received by the Secretary January 29, 1932.

After this experience the rivet has a permanent or "initial" stress of about 19 160 − 9 200 = 10 000 lb., instead of the original 13 410 lb. If the flange is less stiff it is obvious that it will not allow $2S$ to reach 38 320 lb., so the total elongation of the rivet will be slightly less in order to bring it down to its yield point, and its final permanent stress slightly more.

If the flange is stiffer, $\frac{dy}{dx}$ will be less, and, again, the final permanent stress will be more. It seems, therefore, that there is no way to reduce the permanent stress to zero, and loosen the rivet, without carrying the total pull beyond the yield point of the rivets. For this reason it would seem that the factor of safety against loosening the rivets is the ratio of the yield strength of the rivets to the actual pull on the connection.

Another point has to do with Table 3. Consider, for instance, a 15-in., 50-lb. channel, with a 24.2-kip pull (see Fig. 12). Assuming the dimensions shown, the uplift on the first two rivets in the channel web is more than $\frac{24.2 \times 2}{1.7} = 29.5$ kips. The bending in the web of the channel is about 29.5×1.2 in. $= 35.5$ in-kips. With a section modulus of $\frac{12}{6} \times 0.716^2 = 1.02$, the bending stress is 35 kips; also, the direct stress is 2.8 kips per sq. in., making a total tension in the upper surface of the web of about 37.8 kips per sq. in., which is high.

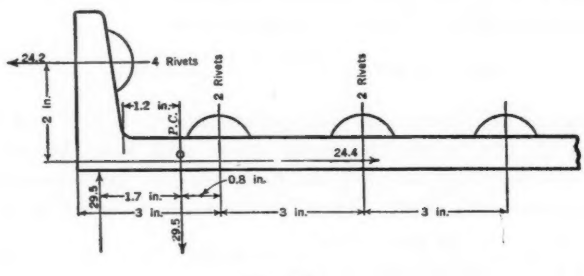

FIG. 12.

These rough figures indicate that 4-rivet angle or channel connections are likely to have such high stresses in tension in the first pair of rivets and in the horizontal leg that they have little theoretical safety factor. Their greatest use is on jobs which, on account of the shape of the building and the number of walls and partitions, need no wind-bracing, except to comply with legal requirements.

Beam stub connections, therefore, would seem to be stronger, and angle connections weaker, than is claimed by the author. This fact does not in any way detract from his main point, the comparative lack of stiffness in top-and-bottom connections, and the immense superiority of the knee-brace.

WILLIAM R. OSGOOD,[6] ASSOC. M. AM. SOC. C. E. (by letter).[6a]—The author points out a sore spot in structural practice, which well deserves the attention he gives it. The fact that premature failures of many connections of the kind he discusses do not occur, may be attributed to the "cleverness of the material"[7] in adjusting itself to conditions which overstress it. The reserve of strength in such connections is necessarily very small.

It is interesting to apply Mr. Berg's method of analysis to the tests made by C. R. Young, M. Am. Soc. C. E., at Toronto, Ont., Canada, in 1927.[8] After

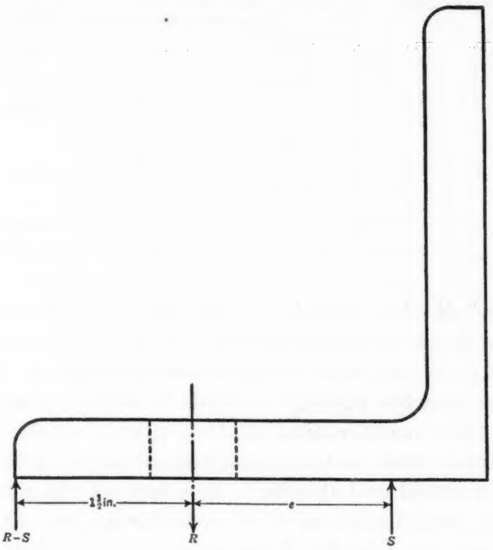

FIG. 13.

the initial tension in the rivet has been exceeded, the force, *R-S,* begins to move outward; and at failure, if the material is stiff enough, this force will have moved close to the outer edge of the connection, as shown in Fig. 13. Professor Young's test angles were free to move sidewise without any restraint on the outstanding leg, so that the values of e and X do correspond.[8a] From Fig. 13:

$$S = \frac{1.5\,R}{e + 1.5} \quad \text{.........................(26)}$$

and if this equation is used to compute the strengths of the rivets in Professor Young's tests, the results in Table 4 are obtained, with a factor of

[6] Engr., Testing Materials, U. S. Bureau of Standards, Washington, D. C.

[6a] Received by the Secretary February 11, 1932.

[7] "Dynamische Bruchversuche mit Flugzeugbauteilen, von Heinrich Hertel, *Zeitschrift für Flugtechnik und Motorluftschiffahrt,* Munich, September 14, 1931. Hertel says "Schlauheit des Materials."

[8] "Tensile Working Stress for Rivets," by C. R. Young, Address before the Fifth Annual Convention, Am. Inst. of Steel Construction, Inc., Pinehurst, N. C., October, 1927; also, *Engineering and Contracting,* December, 1927.

[8a] *Proceedings,* Am. Soc. C. E., January, 1932, p. 7, Lines 6 and 7.

safety of four. The agreement between the computed values and the test values is better than that obtained by either Professor Young's formula or the formula proposed by the writer.[9]

TABLE 4.—COMPARISON OF FORMULA FOR PERMISSIBLE TENSILE STRESS ON RIVETS, WITH RESULTS OF TESTS

University of Toronto, 1927	MINIMUM TEST VALUES DIVIDED BY FOUR, IN POUNDS PER SQUARE INCH — Diameter of Rivets Before Drive			Permissible tensile stress by formula, in pounds per square inch
	$\frac{5}{8}$-in.	$\frac{3}{4}$-in.	$\frac{7}{8}$-in.	
	16 460			.....
Pure tension, $e = 0$		17 110		.....
			13 790	.....
Tension and Flexure:				
	9 690			9 410
$e = 1\frac{1}{8}$ in.		9 750		9 780
			8 260	7 880
	6 550			6 580
$e = 2\frac{1}{4}$ in.		7 200		6 840
			5 790	5 520

O. G. JULIAN,[10] M. AM. SOC. C. E. (by letter).[10a]—This paper is an able presentation of a much neglected subject. It is believed that too often the stresses resulting from eccentricity of connections are given insufficient attention and that at times the rigidity of joints is neglected altogether.

It is clear without mathematical analysis that if all the joints of a rigid frame rotate through equal and reasonably small angles no error in the computed moments is introduced thereby. However, if the relative rotation of the joints is other than unity, errors of considerable magnitude in the computed moments may be introduced, because the less rigid connections and portions of the main members that intersect at a joint are relieved of, and transfer moments to, the more rigid portions of the frame. Tests by Professors W. M. Wilson and H. F. Moore[11] indicate that the relative rotation of well-designed rigid joints equipped with brackets, may introduce errors of about 8% in moments computed on the basis that the connections prevent, entirely, the relative rotation of the main members intersecting at a joint. Furthermore, the relative rotation of joints built without special consideration as to absolute rigidity may cause large errors in the computed moments. Due to deformation of the connections, the total deflection of a rigid frame equipped with well-designed and bracketed joints may be increased by as much as 20%, while the total deflection of frames equipped with strong but comparatively less rigid joints may be more than doubled. It is apparent that, in the design of rigid frames, the absolute and relative rigidity of the connections should be given careful consideration, as well as the stresses.

[9] *Engineering and Contracting,* March, 1928.

[10] Engr. with Jackson & Moreland, Boston, Mass.

[10a] Received by the Secretary February 13, 1932.

[11] "Tests to Determine the Rigidity of Riveted Joints of Steel Construction," by Wilbur M. Wilson, M. Am. Soc. C. E., and Herbert F. Moore, *Bulletin No. 104*, Eng. Experiment Station, Univ. of Illinois.

The author introduces the important but controversial question of the use of rivets in tension. Many specifications do not mention tension rivets, while some others state that rivets shall not be subjected to applied tensile stress. It appears to the writer that the latter provision places an undue restriction upon the design of structural connections. Tests made at the University of Toronto[12] and at the University of Illinois[13] indicate that tension rivets are reliable. Several recent specifications permit tensile working stress on rivets varying, with the specification, from 30 to 100% of the allowable stress in shear. Tension rivets are practically always also stressed in shear. Some authorities claim that the direct shearing and tensile stresses are independent of each other and need not be combined. C. R. Young, M. Am. Soc. C. E., suggests the following tentative formula for rivets in combined tension and shear:[14]

$$p_t = 21\,000 - 8\,000d - 6\,750 \left(\frac{V'}{T'}\right)^2 \ldots\ldots\ldots\ldots(27)$$

in which, p_t is the permissible tensile stress on rivets, in pounds per square inch of rivet before driving; T', the total tension on the rivet due to all causes; V', the total shear on the rivet due to all causes; and, d, the diameter of the rivet, in inches, before driving.

Equation (27) is based on a factor of safety of 4 with respect to the ultimate strength, and was devised to take account of variations in the ratio of V' to T' from zero to 0.85, the latter being the highest value covered by the tests.

The tensile stress in a rivet must of necessity be at least equal to the initial stress due to driving and cooling. It has been claimed by some that the applied tension is additive to this initial tension. However, both theoretical considerations and tests show that if fillers are not used between the active parts, this view is fallacious,[15] and that the initial tension is not increased by an applied tension of lesser magnitude. If fillers are used between the active members, an applied load, P, which is less than the initial tension, increases the total tensile stress to,

$$\frac{R}{A_R} = \frac{R_i}{A_R} + \frac{P}{A_R + A_p} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(28)$$

in which, A_R is the area of the rivet; and, A_p, the effective area of the plate compressed by the rivet in question.[16]

It is believed that in most cases, $A_R + A_p$ may be taken as equal to the area of the plate (normal to the axis of the rivets) between adjacent rivets

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[12] "Permissible Stresses on Rivets in Tension," by C. R. Young, M. Am. Soc. C. E., and W. B. Dunbar, *Bulletin No. 8*, Section No. 16, Univ. of Toronto.

[13] "Tension Tests on Rivets," by Wilbur M. Wilson, M. Am. Soc. C. E., and William A. Oliver, Assoc. M. Am. Soc. C. E., *Bulletin No. 210*, Eng. Experiment Station, Univ. of Illinois.

[14] *Bulletin No. 8*, Section No. 16, Univ. of Toronto, Formula 6, p. 418.

[15] "Permissible Stresses on Rivets in Tension," by C. R. Young, M. Am. Soc. C. E., and W. B. Dunbar, *Bulletin No. 8*, Section No. 16, p. 405, Univ. of Toronto; also "Tension Tests of Rivets," by Wilbur M. Wilson, M. Am. Soc. C. E., and William A. Oliver, Assoc. M. Am. Soc. C. E., *Bulletin No 210*, Eng. Experiment Station, Univ. of Illinois, Section (6) of "Conclusious," p. 35.

[16] *Engineering News-Record*, June, 1928, Vol. 100, No. 23, p. 903.

or, in the case of isolated rivets, as at least 9 sq. in. Then, for a $\frac{7}{8}$-in. rivet, with $\dfrac{R_t}{A_R} = 26\,000$ lb. per sq. in., and an applied load, P, of 15 600 lb. (26 000 lb. per sq. in.): $\dfrac{R}{A_R} - \dfrac{R_t}{A_R} = \dfrac{15\ 600}{9} = 1\,730$ lb. per sq. in.; that is, an applied load equal to the initial tensile force on the rivet in this case increases the tensile stress only 6.7 per cent.

It has been shown[17] that the limiting (elastic limit or yield point) stress of members subjected to direct shear (by applying torsion) is approximately 60% of the limiting stress in tension or compression. Furthermore, it has

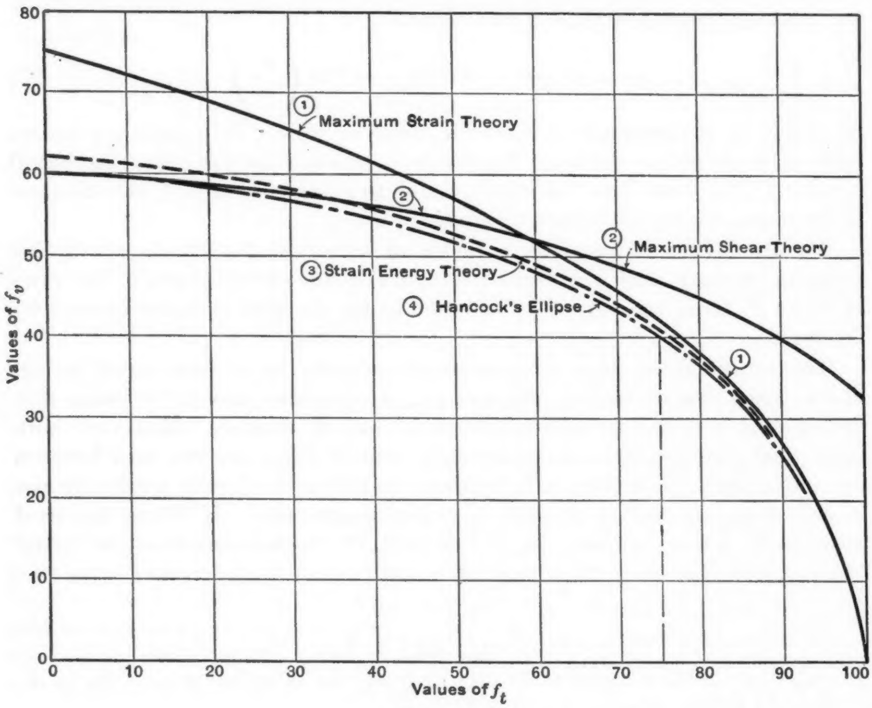

FIG. 14.

been shown that the limiting stress produced by simultaneously applying tension and shear at right angles to each other, may be determined according to the maximum strain theory up to the point where the limiting stress according to the maximum shear theory[18] (with the limiting stress in shear taken as 60% of that in tension), is reached. Beyond that point the maximum shear theory applies. The limiting stresses according to these two

[17] "The Strength and Stiffness of Steel Under Biaxial Loading," by Albert J. Becker, *Bulletin No. 85*, Eng. Experiment Station, Univ. of Illinois.

[18] *Loc. cit.*, p. 8 *et seq.*; also, "Strength of Materials," by Dr. S. Timoshenko, Pt. II, p. 705 *et seq.*

theories are represented graphically by Curves (1) and (2) of Fig. 14. The equations of the curves in this diagram are, as follows:

Curve (1), the maximum strain theory:

$$f_v = \tfrac{1}{4}\,(90\,000 - 600\,f_t - 3\,f_t^2)^{\frac{1}{2}} \quad \text{.............. (29)}$$

Curve (2), the maximum shear theory:

$$f_v = \tfrac{1}{2}\,(14\,400 - f_t^2)^{\frac{1}{2}} \quad \text{........................ (30)}$$

Curve (3), the strain-energy theory:

$$f_v = 61.2\left[1 - \left(\frac{f_t}{100}\right)^2\right]^{\frac{1}{2}} \quad \text{...................... (31)}$$

Curve (4), Hancock's ellipse:

$$f_v = 60\left[1 - \left(\frac{f_t}{100}\right)^2\right]^{\frac{1}{2}} \quad \text{.................. (32)}$$

in which, f_t is the limiting tensile stress in percentage of limiting stress in direct tension; and f_v is the limiting shear stress, in percentage of limiting stress in direct tension.

Curve (2) has been drawn with the limiting stress for direct shear taken as 60% of the limiting stress for direct tension. Curve (3) indicates the limiting stress computed according to the strain energy theory.[19] In determining Curves (1) and (3), Poisson's ratio has been taken as one-third. The writer believes that, for practical purposes, Curves (1), (2), and (3), of Fig. 14 may be replaced by Hancock's ellipse[20] which is shown by Curve (4). The application of this curve may be illustrated by the following example: If a member is stressed to 75% of the limiting stress in direct tension, an additional shearing stress, amounting to 40% of the limiting value in direct tension, will result in the total limiting stress being reached. (See dashed lines on Fig. 14.)

It is evident that all properly driven rivets are subject to initial tension. The magnitude of this tension for hot-driven rivets has been found to be at least 70% of the yield point strength of the rod from which the rivets are manufactured[13] (average, 37 184 lb. per sq. in.); that is, the initial tension is at least 26 000 lb. per sq. in. From the foregoing considerations, it would appear that the working stress in shear should be limited to such a value as to insure that the elastic limit in combined tension and shear is not exceeded. Fig. 15 is based on Hancock's ellipse and is drawn for a limiting tensile stress of 32 kips per sq. in. (which is assumed to be the elastic limit), and a limiting shearing stress of 60% of this value; that is, 19.2 kips per sq. in. The equation of this ellipse is,

$$f_v = 19.2\left[1 - \left(\frac{f_t}{32}\right)^2\right]^{\frac{1}{2}} \quad \text{..................... (33)}$$

[19] "Strength of Materials," by Dr. S. Timoshenko, Pt. II, p. 708.

[20] "Results of Tests of Materials Subjected to Combined Stress," by E. L. Hancock, *Proceedings*, Am. Soc. for Testing Materials, 1908, p. 373 *et seq.*; also, "The Strength and Stiffness of Steel under Biaxial Loading," by Albert J. Becker, *Bulletin No. 85*, Eng. Experiment Station, Univ. of Illinois.

in which, f_v is the limiting shearing stress, in kips per square inch, and f_t, the limiting tensile stress, in kips per square inch.

Based on Equation (33) and for an initial tensile stress of 26 000 lb. per sq. in., the working stress in shear should not exceed 11 200 lb. per sq. in. (see dashed lines on Fig. 15), which is somewhat lower than the value allowed by standard specifications.

Regarding the connection shown by Fig. 2 it is evident that the rivets above the neutral axis of the connection between the hitch angles and the column are in tension as well as in shear. However, it is not clear that this neutral axis coincides with that of the girder itself. Four texts[21] consulted by the writer on this point give three materially different answers. It would appear that this question is not as simple as it is usually represented, the

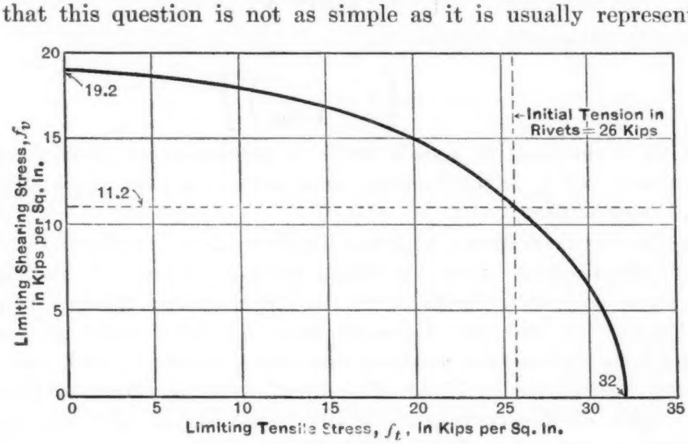

FIG. 15.

location of this neutral axis and, hence, the tensile stress in the rivets, being dependent upon the rigidity of the bearing surfaces, the initial tension in the rivets, and the axial load on the girder. It is believed that Mr. Berg's views on this question would be interesting and enlightening.

The author mentions the "prevalent practice of neglecting the gravity loads." It is difficult to understand how such a practice can be defended or how it became prevalent. Standard textbooks[22] give methods of determining the moments resulting from gravity as well as from wind loads, and the corner moments resulting from gravity and wind loads can be determined by means of the Cross method of distributing fixed-end moments.[23] It appears obvious that frames should be proportioned so that when the structure is subjected to the most unfavorable reasonable combination of dead, live, and

[21] "Strength of Materials," by the late George Fillmore Swain, Past-President and Hon. M. Am. Soc. C. E., p. 392 *et seq.*; "Structural Engineers' Handbook," by Milo S. Ketchum, M. Am. Soc. C. E., Third Edition, p. 730; "Structural Members and Connections," by Hool and Kinne, Section 3, by C. A. Wilson, Assoc. M. Am. Soc. C. E., p. 339; and "Design of Wind Bracing," by Edward Smulski, *Journal,* Boston Soc. of Civ. Engrs., Vol. XVII, No. 9, p. 509 *et seq.*

[22] For example, "Modern Framed Structures," by Johnson, Bryan, and Turneaure, Tenth Edition, Pt. II, Section V, p. 530 *et seq.*

[23] "Analysis of Continuous Frames by Distributing Fixed-End Moments," by Hardy Cross, M. Am. Soc. C. E. *Proceedings,* Am. Soc. C. E., May, 1930, Papers and Discussions, p. 919.

wind loads, no part, including details and rivets, will be stressed beyond the endurance limit for the range of stress to which the part is subjected. The endurance limit for 0.2% carbon steel subjected to complete reversal of stress has been shown to be about 25 000 lb. per sq. in.[24] Dr. Timoshenko gives a corresponding limit of 27 000 lb. per sq. in. for mild steel.[25]

A number of interesting and useful formulas have been presented by the author. It is believed that their value would be considerably enhanced by the amplification of their derivations.

HAROLD S. RICHMOND,[26] ESQ. (by letter).[26a]—Much of value has been brought out by the author in emphasizing the important problems involving rivets in tension acting through the thin legs of lug-angles. The point illustrated in Fig. 4(*b*) cannot be over-emphasized, that is, in a tension connection only those rivets immediately adjacent to the pulling stem of the connecting lug are effective. In a case sometimes encountered, two such lugs or diaphragms are opposed to each other on opposite sides of a web-plate and at right angles to each other. In such a case, it is obvious that only those rivets nearest the intersection are effective in tension.

In dealing with this class of problem the writer makes a distinction between static loads and variable or reversible loads and also between self-relieving stresses and those of a definite nature. For example, if the connection illustrated in Fig. 2 is considered to be merely a shear connection and not of use as a bracing connection, then thin lug-angles will make those stresses resulting from static loads self-relieving, but the stresses resulting from variable or reversible loads will result in fatigue unless held within the elastic range. Mr. Berg's comment that "the computed stresses in the angles are high, but harmless," therefore, will be taken as safe counsel only to the extent that the detail is subject to deflection from static loads. The danger in this case is that the design when put into use seems to be a successful one; yet the structure cannot be considered permanent unless all variable stresses lie within the elastic range. The writer sometimes satisfies his conscience by considering that the variable-load stresses are self-relieving to the extent of ranging themselves between a positive and a negative stress of equal magnitude after self-adjustment, rather than between zero and a higher stress, but always of the same sign.

In the case encountered by the writer, of a girder connection in a railroad viaduct, similar to that shown in Fig. 2, the lug-angle was split at the root, due presumably to fatigue. Whether or not the structure was built of wrought iron or steel, the criticism will apply with sufficient force, and designs of this kind will call for extremely careful calculation should wrought iron ever come back into vogue as a structural material.

While it is important to calculate and limit the tension in the rivets, it is at least equally important to limit the bending stresses in the legs of the

[24] "A Study of Slip Lines, Strain Lines, and Cracks in Metals Under Repeated Stress," by Herbert F. Moore and Tibor Ver, *Bulletin No. 208*, Eng. Experiment Station, Univ. of Illinois.

[25] "Strength of Materials," by Dr. S. Timoshenko, Pt. II, p. 682.

[26] With Gibbs & Hill, Cons. Engrs., New York, N. Y.

[26a] Received by the Secretary February 17, 1932.

connection lugs in the case of variable stresses. If the author's calculations are used, the tension in the rivet will range between the initial stress and the yield point—a range of only 11 100 lb. per sq. in.—and this only, needs to be considered in relation to fatigue effects, whereas, the variable bending stresses in the lugs will usually run much higher. There is, of course, a bending stress in the rivet due to the "ripping" effect of the lug. The theory would be difficult to develop and designers might possibly assume that this bending stress is partly self-relieving.

In brief: Connection lugs of thin material are indicated for constant loads, while thick material with the design carefully analyzed to keep stresses within the elastic range is indicated for variable loads. There would be danger of overstressing the rivets in the case of high moments due to constant loads and danger of fatiguing connection lugs in the case of variable loads.

ROBINS FLEMING,[27] ESQ. (by letter).[27a]—By his painstaking paper Mr. Berg has again brought to the attention of structural engineers the importance of the proper design of wind-bracing connections. In addition to Mr. Berg's paper three others lie before the writer, namely: "Design of Wind Bracing," by Edward Smulski;[28] "Connection Angles Subjected to Bending," by Norman B. Green;[29] and "Details for Wind Connections in Tier Buildings," by Walter H. Weiskopf,[30] Assoc. M. Am. Soc. C. E.

The opinions of these writers and those of other engineers show at times a variation of 100% in the strength of connections. In actual practice the variation is even greater. It would thus seem that tests are necessary before definite and convincing conclusions can be reached. In fact, the theories put forth from any set of assumptions should be substantiated by a series of full-sized tests covering a wide range of connections as well as a number of connections of the same type.

Mr. Berg states,

> "The insecurity in the present method of rivet design has been illustrated, showing particularly how a stress of 18 000 lb. per sq. in. on the rivet as used by many designers, may develop stresses actually twice that amount."

Others will doubtless examine, critically, the reasoning that leads to such a sweeping claim. The writer will call attention to a few points.

Mr. Berg locates the point of contraflexure by the theory of flexure. In his analysis of the flange of a 24-in. 120-lb. beam, he assumes a span of $2\frac{1}{2}$ in. and an average depth of about $1\frac{1}{4}$ in. The theory of flexure is scarcely applicable to a beam of these proportions. Moreover, the deformation from shear would be nearly as large as that from flexure.[31]

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[27] With The Am. Bridge Co., New York, N. Y.

[27a] Received by the Secretary February 25, 1932.

[28] *Journal,* Boston Soc. of Civ. Engrs., Vol. XVII, p. 491, November, 1930.

[29] *Western Construction News,* Vol. 5, p. 137, March 10, 1930.

[30] *Engineering News-Record,* Vol. 99, p. 396, September 8, 1927.

[31] "Horizontal Shear and Shearing Deflection of Beams," by Robins Fleming, *Engineering News-Record,* Vol. 107, p. 896, December 3, 1931.

Mr. Berg refers to a paper by Professor Young and states, "* * * it becomes important to use the right judgment relative to the eccentricity, e (Fig. 4(a))." In the paper to which reference is made[32] Professor Young states,

"In the eccentric loading tests, the rivet connecting the cross-piece to the longitudinal member was tested by applying the load to the cross-piece at a distance of $1\frac{1}{8}$ and $2\frac{1}{4}$ in. from the gage line of the angles by means of knife-edges attached to the ends of the inverted **U**-shaped castings."

This clearly defines e as the distance from the center of the rivet to the knife-edge or point of contraflexure, which is the same as that defined by Mr. Berg. The author, however, takes e as $\frac{X}{0.65}$, which is too large. This changes the results of his Table 2.

Sometimes e is taken as $\frac{1}{2A}$, A being the distance from the face of the web to the center line of the rivet. This, however, assumes that the vertical leg is fixed at the root. With thin flange angles or channels there is a tendency to bend about the rivets in the horizontal leg.

The question of rotation from deformation of connections should be determined from tests. The structural engineer is not often concerned about "Behavior Beyond the Elastic Limit." His efforts are directed to keeping stresses within the elastic limit. Nevertheless, it is well the subject should be understood. Mr. Berg has written an able paper but experimental data are needed to confirm his results. In conclusion, a valued correspondent will be quoted. He writes:

"The only wind detail that has been used up to date that is susceptible of an exact mathematical analysis is the detail where there was a common inserted web plate between the column and the wind girder. This detail has not been used for a long time it having for commercial reasons passed out of the picture."

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[32] "Tensile Working Stresses for Rivets Investigated," by C. R. Young, M. Am. Soc. C. E., *Engineering News-Record*, Vol. 100, p. 128, February 2, 1928.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

PUBLIC SUPERVISION OF DAMS
A SYMPOSIUM

Discussion

BY MESSRS. WILLIAM P. CREAGER, M. M. O'SHAUGHNESSY, N. A. ECKART, R. C. JOHNSON, F. W. HANNA, AND JOEL D. JUSTIN

WILLIAM P. CREAGER,[9] M. AM. SOC. C. E. (by letter).[9a]—The writer desires to compliment the authors on their meritorious presentation of the subject. Their ideas and compilations will be of immeasurable assistance in connection with the work of the Committee of the Power Division on Legislation Respecting Safety of Dams in preparing a typical draft of a law for the regulation of the construction and maintenance of dams.

Messrs. Markwart and Hinderlider have covered the subject so completely as to leave little opportunity for further constructive data. There remains, then, only the opportunity for an expression of opinion in the matter of details. The writer thoroughly agrees with Mr. Markwart's statement under "Burden of Cost" that,

> "While every consideration should be given to meritorious complaints [of danger], the State must safeguard itself against nuisance complaints and must not permit itself to become an instrument of malicious persecution."

However, his next statement to the effect that "to this end many States require deposits (returnable) of complainants * * *," if taken literally would seem to indicate that, in such cases, the possibility of a property owner obtaining immediate and adequate police protection in this respect is contingent upon his possession of liquid assets. The only cases, to the writer's knowledge, in which such deposits are required are when the complaint, after adequate gratuitous investigation by the State agency, is considered by it not to be meritorious and the complainant demands further investigation or inspection.

It is the evident desire of all laws governing the supervision of dams to provide for State approval in all cases where failure would involve loss of life and property and to exempt those which do not. It is fallacious to make

NOTE.—The Symposium which includes the paper by A. H. Markwart, M. Am. Soc. C. E., presented at the meeting of the Power Division, New York, N. Y., January 16, 1930, and the paper by M. C. Hinderlider, M. Am. Soc. C. E., presented at the Technical Session, Sacramento, Calif., April 23, 1930, respectively, was published in January, 1932, *Proceedings*. Discussion of the paper has appeared in *Proceedings* as follows: March, 1932, by H. deB. Parsons, M. Am. Soc. C. E.

[9] Chf. Engr., The Power Corporation of New York, Buffalo, N. Y.

[9a] Received by the Secretary January 25, 1932.

the assumption, as has been done in many statutes, that a dam having a certain height or a certain size of pond is on the border line between peril and security, omitting entirely the consideration of length and other important influences.

The damage done by a flood is a function of the velocity, the depth, and the duration of the discharge. The first two factors, ordinarily, have the preponderating influence on the extent of destruction. For a given channel below the dam, the velocity and depth are determined by the rate of discharge. The rate of discharge is governed not only by the height of the dam, but also by the width of the break, which is frequently the length of the dam. Furthermore, in addition to the characteristics of the dam, the degree of utilization and occupancy of the valley below it has, in many cases, a great influence. It seems difficult, if not impossible, therefore, to fix definite limits on the size of dams not subject to State approval.

If thought to be feasible, it might be desirable to require the approval of all dams of any size or nature, or perhaps those limited to an unusually small minimum size; but with the provision that, if in the opinion of the owner the failure could not possibly cause loss of life or property, a preliminary application could be presented accompanied only by sketches and written descriptions. The State agency would then notify the owner whether or not a final application in prescribed form would be required.

The writer believes that, with the provision of appeal only to the Courts from the decision of the State agency, Mr. Markwart's hearing "at the earliest possible time" would ordinarily not be soon enough. The writer is inclined to favor his alternative provision for presentation of the case to a board of engineering arbitrators. However, there still remains the question as to whether the provision of an appeal to the Courts from the decision of the board would not be justified.

If it could be proved to be practicable, the writer is very much in favor of a code. However, it is a lamentable fact that the divergence of opinion among engineers respecting the proper design of dams is so great as to preclude the possibility of universal agreement. On the other hand, without a code for the guidance of the State agency, the requirements for approval of design might be altered radically at each change in tenure of such agency. This condition might seriously affect the possibility of making accurate estimates of cost.

A complete code in the sense used in building construction would require several hundred pages. In fact, it would be a treatise on the design and construction of dams, with explanations and proofs of theory omitted. The new California regulations allot about three pages to "General Information" which is, in effect, a miniature code. One cannot deny that it is a help, but to such an insignificant degree as to cause the writer to wonder why it was included. It is believed that a code will not be feasible until engineers have standardized the theory of design to a much greater extent.

The writer is of the opinion that there will be little or no opposition to State regulation in all States. Owners of dams should welcome the opportunity for an independent review of their engineering designs at a cost much less than is customarily paid for consulting services.

M. M. O'SHAUGHNESSY,[10] M. AM. SOC. C. E. (by letter).[10a]—The St. Francis Dam disaster in Southern California was a tragic event, which has created an hysteria on dam construction in California and the West. This catastrophe was of such an appalling extent that citizens have practically lost their heads on the subject of control of dam design and construction and have enlisted the aid of the Legislature of the State of California to pass a most drastic law covering the subject.

The writer was personally opposed to the passage of this law in the interest of the City of San Francisco, because he did not desire to see State authorities intrude themselves into local projects. On October 3, 1928, he protested to the State Engineer against this legislation, making the statement that after every disaster to a dam there is great hysteria among the newspapers and the public to vest supreme authority in some outside source so as to guarantee the safety of all future structures. The great problem is to find that "some one" who has gathered enough wisdom from his experience and who has adequate force, technical knowledge, and authority to fill the job. The commission of sins regarding dam building will not be cured by delivering the problem into the hands of young Civil Service employees who may pass adequate examinations after coming freshly from universities, qualifying them to act as aids to the State Engineer.

The amended Act controlling dam construction in California has adequate provision for supervising this work. In some ways the Act is unfair because the owner of the property may not submit his engineer, as a member of the supervising board vested with authority, to pass on the engineering structure. That responsibility is entirely assumed by the State Engineer and his assistants. Hence, unless the field of the State Engineer is well established, engineers on dam construction are precluded from rendering valuable service in designing large industrial projects.

Mr. Hinderlider's recommendation for a revision of the code will safeguard the construction of dams. Due care should be exercised in formulating this code because a very unhappy solution might result if an undesirable code is adopted, such as the Italian regulations governing the construction of rock-fill dams. The suggestion that engineers should study the problem of directing public opinion along sane lines that would lead to constructive measures for their protection, is a good one.

The Society is indebted to Mr. Markwart for his clear exposition of the problem and also to Mr. Hinderlider for the exposition of practice in Colorado. The Society, through the engineering press, should not hesitate to expose the construction conditions underlying all dam failures, as it is only by an intense study of those disasters that safety in all future dam building will be developed. The best account of dam failures that the writer has seen is included in a paper by Mr. F. C. Uren, entitled "The Design of Reservoir Dams, with Some Accounts of Failures."[11]

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[10] Cons. Engr., San Francisco Public Utilities Comm., San Francisco, Calif.
[10a] Received by the Secretary February 1, 1932.
[11] *Transactions*, Institution of Civ. Engrs. of Ireland, Vol. XXXVI, 1910, p. 52.

It is not unlikely that defective dams are fewer in number than defective buildings. On being filled with water they are very quickly brought to a supreme test of endurance and their weaknesses exposed, whereas defective buildings may survive a long time without being discovered. Great credit is due to the American engineers in the past thirty years who have developed the art of safe building of high dams with real fortitude and discretion.

N. A. ECKART,[12] M. AM. SOC. C. E. (by letter).[12a]—The subject of this Symposium is one that should induce a widespread discussion, because the Engineering Profession is somewhat divided as to the desirability of having the supervision and authority over the design, construction, and operation of all dams in a State vested in a public official or body. Those who advocate State supervision will differ among themselves as to the extent of the authority to be vested in that official, and those who oppose it will express many varied and worthy arguments therefor.

Much of the value of Mr. Hinderlider's paper lies in the fact that the author has set forth concisely his views as to what State supervision should include, its advantages and disadvantages, and the qualifications which the official charged with supervisory control should possess.

The question as to whether or not supervision of dams should be vested in the State Engineer, or other public official, is not one which will be decided by the Engineering Profession, and any discussion of that phase of the subject would be more or less academic. The right of the State to vest supervision and control of dams in the State Engineer can not be questioned as a part of its police power looking to the public safety, and legislation providing for such supervision no doubt in most instances is the result of a public demand following some disaster.

The Engineering Profession, however, is deeply concerned in the form such legislation will take, the extent of the authority vested in the State official, the manner in which the authority is exercised, and the procedure involved, but even more than in the form of legislation practising engineers are interested in the individual who is chosen to exercise this power, not only as to his qualifications from the viewpoint of training and technical ability, but even more as to his executive ability, the broadness of his view, and the soundness of his judgment, or, as the author terms it, the amount of his "horse sense." The right man in charge of State supervision can make almost any legislation workable and can secure the desired results in soundness of design and construction, with the minimum of interference with the work of competent engineers. The wrong man can play havoc with the best engineering plans, construction methods, and program; petty interference, incompetence, indecision, or delay in approving proper plans or foundations, may add hundreds of thousands of dollars to the cost of a project, as the delay of a few weeks at some stage may result in throwing the work into the flood period and possibly may cause the loss of a season's run-off. In an Eastern State, approval of the design for an arch dam was withheld, pending the pub-

[12] Gen. Mgr., San Francisco Water Dept., San Francisco, Calif.

[12a] Received by the Secretary February 1, 1932.

lication of the report of Engineering Foundation on the "test dam" at Stevenson Creek.

The California law has not been in effect long enough to permit judgment to be passed as to the workability of the procedure, but it has gone into effect under the most favorable auspices as to personnel. The Engineering Department has a large task ahead in reviewing the design and construction of all the dams in the State (some of which were built 70 or 80 years ago), in addition to checking the designs and construction of current work.

The greatest advantage of State supervision is in insuring disinterested and independent review of all dam design and foundation conditions and independent inspection during construction. The value of this independent check is dependent on the competence of the reviewer. If he is incompetent the review is more than worthless, for, in many cases, it will result in preventing a competent review and check by independent consulting engineers and geologists who otherwise would have been called in. It behooves engineers to use their best endeavors to have the high standard of the officials maintained.

As the first advantage Mr. Hinderlider cites "centralized authority with singleness of purpose and responsibility to the public." This is only realized in part in California since dual authority, as between the State Engineer and the Federal Power Commission, still exists in connection with projects involving the use of lands in the Federal Reserve. This is a condition which should be corrected, either by delegation of authority or adoption of uniform requirements and standards.

Another important advantage is the constructive work that will be done by the State departments toward the development of a code of practice in the design, analysis, and construction of dams. This will be done as a matter of convenience and necessity for the uniform administration of the law with respect to these matters. With the vast number of projects coming before the Department for review and check, a uniform procedure must be adopted, with standard requirements as to factors of safety, methods of analysis, and principles of design. The adoption and promulgation of such a code of practice by the State department will avoid the necessity of much delay and unnecessary work in re-design and re-submission of plans, which might be necessary to meet the requirements of the Department, and which might not otherwise be definitely known.

The writer can not agree as to all the advantages claimed for State control noted in Mr. Hinderlider's paper. There is no assurance, for instance, of the minimum effects of local and political influence; this is no doubt true as between county and State control, but it does not hold as between State control and no State control. Co-ordinated control in administration of the uses of stored water and stream flow does not necessarily go with the State supervision of dams. Such power might be considered as an advantage for the general public interest, but might result in serious detriment to the development of the resources of a State; it would depend largely on how it was administered. Undoubtedly, the disadvantages of State control, as set forth, will not meet with any disagreement.

As to the qualifications of the official charged with State supervision of dams, in addition to those which Mr. Hinderlider has specified, the official should have a good fighting heart, the ability to stand on his feet and forcibly and clearly express himself to defend his views, and, above all, he should have the moral courage to accept full responsibility for the failures as well as the successes in his administration of the office.

R. C. Johnson,[13] Assoc. M. Am. Soc. C. E. (by letter).[13a]—The feasibility of a building code for dams is discussed in this Symposium. Not only is such a code feasible, but it would be practicable, a great aid to safer designs, and a very definite step toward an ultimately uniform practice in building dams.

No doubt such a code would of necessity be very involved, due to complications of conditions and different types of structures to be considered; but this would not make the undertaking impossible. At present, a great number of differences of opinion exist in relation to certain features of design. Uplift pressure under masonry structures, expansion and contraction joints, and the distribution of stresses in gravity arch dams, are only a few of the important features actually considered in a variety of ways. The existence of this diversity of opinions alone offers sufficient grounds for the formulation of a code, however difficult that may be. Ultimately, codification would also result in a solution of many of the existing problems and uncertainties. Authenic investigations and tests would, of necessity, be made.

It is commonly granted that the responsibility for the safety of the structure must rest with the designers, owners, and the Engineering Profession in general. It is of paramount importance, therefore, that the Engineering Profession do all within its power to perfect such safety. The formation of a code to promote uniform design and construction should be equally as important as the promotion of legislative action. Should the designers and owners themselves be able adequately to provide for the safety of such structures, no further State legislation and regulations would be necessary.

The writer agrees with Mr. Hinderlider in that a building code enacted as law does not appear practicable at the present time, due to the complexity of the problems involved and the subsequent continual changing that will be necessary. Nevertheless, no doubt, such a guide would prove valuable to State engineering staffs engaged in checking and approving general designs.

F. W. Hanna,[14] M. Am. Soc. C. E. (by letter).[14a]—The early establishment of sound legislative and engineering principles in the laws, rules, and regulations governing the public supervision of dams is most necessary. Unless sound principles are included in the original laws many unwise and unjust acts are certain to result.

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[13] Associate Prof., Civ. Eng., Univ. of South Carolina, Columbia, S. C.

[13a] Received by the Secretary February 1, 1932.

[14] Chf. Engr. and Gen. Mgr., East Bay Municipal Utility Dist., Oakland, Calif.

[14a] Received by the Secretary February 3, 1932.

All jurists, and indeed all engineers, must agree with the authors that the State, in assuming supervision over dams, is merely exercising one of its police functions in the interest of the public. The States of the Union have been slow in proclaiming authority over the design, construction, and maintenance of dams, because the necessity for it has not been so apparent, until increase in population and in construction of dams, coupled with recent failures, has made it evident. The danger now is that the pendulum may swing from no control to superfluous control, which will result in loss of time and money to investors and, consequently, to the public in general. This, the Society and the Engineering Profession in general, can help to prevent by proper consideration, action, and advice.

The first essential is to secure State statutes of the proper character, which is not an easy task. The statute granting authority for the supervision of dams should not only be simple, clear, and specific, as should all statutes, but it should be economically reasonable and comprehensive. It is essential, of course, that the act be drawn in simple and clear language to avoid endless disputes and lawsuits, and that it be specific as to what is to be supervised and as to who is to perform the supervision. It should be reasonable in order to prevent unnecessary expenditures in the construction of dams, that would cripple the industries depending upon dam construction. The law must be so comprehensive as to cover all features involved in the safety of the dam, the location, the foundation materials, the design (including outlet works and spillways), the materials of construction, and the method of construction. Unquestionably, the law should also give supervisory control over the maintenance of dams after their construction and should extend to the approval of past as well as to future construction.

In setting up State machinery to handle the act, it is desirable to use existing governmental machinery so far as practicable, but it should be remembered that not all the States of the Union have governmental machinery suitable for the supervision of the construction and maintenance of dams. This supervisory control is essentially and wholly an engineering feature and should not be left to quasi-engineers and politicians. As a matter of fact, the greatest danger of lodging supervisory control of the design, construction, and maintenance of dams in the hands of the State is that of making it a political rather than an engineering problem. Every effort should be made in the statutes and in their control to eliminate politics, professional jealousy, factional disputes, and other evils that are likely to creep into the administration of an act of this character.

The builder of a dam may find it unsatisfactory, in his opinion, to comply with the requirement of the supervisory control, and, there should be opportunity, therefore, for appeal to the Courts or to a board of arbitration. Such a board should be composed of engineers and geologists qualified to pass on the feasibility of the dam and its appurtenances.

In the Western States of the United States, where public lands are still held by the National Government, the Federal Power Commission has authority over dams for power development, and the builder should not be subjected to duplicate authority, nor should he be compelled to thrash out

the differences of opinion between State and National authorities. Where both State and National Governments are interested in any dam, these agencies should either agree on an appropriate division of their authorities, or they should thrash out their differences of opinion and delegate one or the other agency to deal with the builder.

In addition to duplication of authority in intrastate projects, there may be conflicts between interested States on interstate projects on rivers passing through two or more States. A State supervising the construction of a dam may be so fortunate in the location of its population as to make the failure of a dam on an interstate river entirely without danger to persons or property within its boundaries, while the persons and property in an adjoining State through which the river passes may be in dire danger from the failure. In such cases it would seem that the State in which the persons and property are in danger should have supervisory control over the dam constructed in the sister State. Cases of this kind might be quite common since rivers pay little attention to State boundary lines.

The status in interstate problems may be still further complicated in the Western States by the introduction of National authority where federally controlled power projects are designed, constructed, and operated under the approval of the Federal Power Commission. This conflict of interests of neighboring States and National Government may not be an easy one to handle in connection with State supervision of the design, construction, and maintenance of dams. Possibly the solution of the problems in these cases is National supervision. There are also instances both on the northern and southern borders of the United States where not only duplication of State authorities and of State and National authorities may arise, but where duplication of National authorities may be involved. Such situations cannot readily be handled by State supervision of dams within their borders, because international questions are involved. Thus, it appears that the supervision of dams by States is not simple, at least in some of the States.

In the matter of the codification of dam-building principles, the writer believes that it is entirely practicable to codify methods of design and construction of dams for specific foundations. However, there are so many diverse foundation materials to be encountered in the construction of a dam that the multiplicity of code requirements to meet all of them scarcely seems practicable. Nearly every foundation involved in the construction of a dam has some peculiar and specific problem that must be solved in order to determine the character of design and the character of construction to be used, and it will be difficult to meet all these conditions fully and properly in the formulation of a code. Therefore, only general principles that are applicable to each type of dam should be included in the code.

JOEL D. JUSTIN,[15] M. AM. SOC. C. E. (by letter).[15a]—Universal State supervision of the design and construction of dams is almost a certainty of the near future, and the writer believes that such supervision is necessary and

[15] Hydro-Elec. Engr., U. G. I. Co., Philadelphia, Pa.

[15a] Received by the Secretary February 8, 1932.

desirable. On the other hand, it is conceivable that it might lead, in some cases, to unnecessary and unproductive expense, or might even so increase the cost of a project that it could not be executed. Furthermore, if State supervision is merely perfunctory or incompetent, the approval of the project by the State authority would be no index as to its safety. It is highly desirable, therefore, that legislation on the subject be of such a nature as to help to secure the safety of the public without placing financial burdens on the owner which might "sink" the project.

Although open to conviction, the writer is doubtful of the efficiency and desirability of a code which, supposedly, would be a series of specifications covering in detail the design and construction of every type of dam on every conceivable kind of foundation. Such a code might even contain attempts to specify the types of design which would be considered permissible for certain specified foundation conditions.

Under the United States system of government, it would probably not be difficult to get the code, but without provision for a personnel to see that it is carried out efficiently. The State engineering authority would have to certify that the design and construction complied with the code. For the State engineering authority to do this honestly would require nearly as great an expense as that incurred for engineering by the owner. Most, if not all, of this additional expense would almost surely be assessed against the owner.

A code would lead to the attempt to standardize. Standardization in design is not applicable to dams to the extent that it is applicable to some classes of building construction, for instance. Any attempt to reach standardization through a code would undoubtedly lead to much unnecessarily costly construction, and would make uneconomic the undertaking of many desirable projects.

The writer, therefore, favors Mr. Markwart's first suggestion, namely, that the State engineering authority should be charged with passing on the safety of the design of the dam as submitted for the given foundation conditions, and should also be charged with passing on the safety of the structure as built. This latter requirement should mean that for important projects the State engineering authority would have an inspection engineer on the job throughout its construction.

Such an inspector should have no authority to give instructions, but should report his observations to the State engineer, who would take such action as might be necessary. He should confer constantly with the owner's resident engineer and bring to his attention all cases of faulty design and workmanship which he notices.

There are often cases in which such State supervision may be of the greatest help to the owner's engineer. Thus, in the case of a project in which the financial backing is not of the highest class, the pressure on the owner's engineer to design a cheap structure is often great and difficult for the average man to resist. If, in addition, the owner is also doing the construction, the position of the owner's engineer may prove most unfortunate. Precautions which, to the engineer, are of the greatest importance, often seem foolish and expensive refinements to the uninitiated and, therefore, may be

omitted in such cases by the owner's orders. In such cases, State supervision proves a veritable Gibraltar of strength to the owner's engineer, enabling him to get a conservatively safe design adopted and constructed with due regard to safety instead of being compelled to resign in order that he should not be connected with an unsound project.

As Mr. Markwart points out, it is necessary that a method of appeal should be provided from the decisions of the State engineering authority. This is necessary because a State authority interested only in safety might insist on structures which, although safe, were not the most economic which could be designed for the particular conditions. Mr. Markwart's suggestion of a board of engineers appeals to the writer as more desirable and less likely to involve delays than an appeal to the Courts. The decision of the board should be final, of course, and the expense of the appeal should be borne by the owner.

Additional laws providing for closer supervision of the design and construction of dams will be passed by the various State legislatures. If the formulation of these laws is not guided by competent engineers, serious errors will be made.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

STEREO-TOPOGRAPHIC MAPPING

Discussion

By Messrs. Theron M. Ripley, O. S. Reading, and Lowell O. Stewart

Theron M. Ripley,[8] M. Am. Soc. C. E. (by letter).[8a]—In the "Synopsis" of his paper Colonel Birdseye states that it is to be considered as covering the aerocartograph and brings out discussions of it and similar instruments. In the "Introduction" to his paper he compares ground surveys with aerial surveys and shows certain advantages of one over the other.

After the field work is done and the negatives are in the office a trained organization is required to produce a proper map from them by the use of the aerocartograph or any other similar instrument. Except war and other emergency work the engineer is interested only in securing, economically, a survey and map of such accuracy that a reasonable preliminary design and estimate can be made therefrom. For final location and construction purposes ground surveys must be made.

Colonel Birdseye covers the questions of topographic detail, such as drainage, culture, etc.; the usual method of "sketched in" features not "under the feet"; and the mapping of "inaccessible areas"; but he makes no statement of the probability of time saving. The time which can be saved will often be the important factor in determining the use of aerial surveys.

A mountain survey, for sixty miles of railroad line, following a creek in a gorge, required six months with a full party, including camping personnel and equipment. This was in the Rocky Mountains where flying and photographic days are many. An aerial map of that creek and its gorge, with the necessary control, could have been put into the hands of the engineer for projection and estimate in one-quarter the time required for the ground survey and, at the present time, at no greater cost.

An exhaustive estimate of the cost of an aerial survey covering a large territory was made by the writer in 1928. It was approximately the same

Note.—The paper by C. H. Birdseye, M. Am. Soc. C. E., was presented at the meeting of the Surveying and Mapping Division, Sacramento, Calif., April 24, 1930. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[8] Cons. Engr., Buffalo, N. Y.

[8a] Received by the Secretary February 4, 1932.

per mile as the cost of ground surveys in the same country. The great advantage of the aerial method was found to be in the time saved and the mass of accurate detail obtained. The estimated personnel to accomplish this work by ground methods was about equal to that estimated for the aerial method, but the time required for the latter was one-third that of the former.

The two preceding examples are from the writer's personal experience and confirm the statement relative to the importance of stressing the time element.

O. S. Reading,[9] Esq. (by letter).[9a]—A remarkably clear and comprehensive statement of stereo-topographic mapping has been presented in this paper. It may well serve to acquaint the profession generally with the subject as well as supply a basis for further study to those especially interested.

It is difficult to convey the power of the stereoscopic method of mapping without some background of experience on the part of the reader, or at least the possession of a simple stereoscope with some suitably arranged stereoscopic pairs of photographs. The writer recalls a pair which he examined in the author's office. The lines of sight of two figures of triangulation across a wooded valley were shown by fine white lines. The data for one of the stations were in error, but enough other stations appeared on the photographs to orient them correctly. The manner in which the lines of sight that were in error intersected at a point above the ground, while the correct lines intersected precisely at the stations, gave a striking demonstration of the power of the stereoscope and the infallibly uniform accuracy of the photographic record. At the same time the unobstructed view of the undulations of the wooded hills showed the advantages of aerial photographs under a stereoscope over the limited views which would have been available to ground methods on such terrain.

The literature of surveying a few centuries ago abounded with statements hailing the plane-table, which permits the drawing of the map in the field with the earth before it as a model instead of the compilation of more or less inadequate notes in the office. Surveyors of the Twentieth Century may well hail aerial photographs and the stereoscope in the same manner. These devices permit the transportation of a model of the earth instead of more or less inadequate maps to any number of offices for the study of problems as they occur. It is no longer necessary to survey and record a vast amount of data in the effort to anticipate all the problems that may arise, or, later, to make expensive extra trips to the field. There is a growing appreciation among engineers of the value of the almost unlimited detailed information given by aerial photographs under the stereoscope as a supplement to the quantitative information of topographic maps and line surveys.[10]

The writer has yet to examine an aerial re-survey which did not bring out some faults in previous ground surveys, due to mistakes in traverses or

[9] Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

[9a] Received by the Secretary February 18, 1932.

[10] "Aerial Survey for Transmission Line Location," by F. G. Dana, *Civil Engineering*, Vol. 1, No. 4, p. 249; also, "Aerial Photographic Survey and Mapping," by Harry Brighton, *Michigan Engineer*, March, 1931, pp. 42–43.

to interpolation and sketching between instrument set-ups and rod-readings. In contrast, carefully controlled and plotted aerial photographs are as self-checking as a layout of quadrilaterals in triangulation. If the detail is carefully traced all the well-defined objects of a photo-survey will be shown as accurately on the map as the control stations. This uniform precision of detail is particularly valuable for basic surveys intended for use in the general development of an area. The amount of detail that is traced from the photographs guarantees an ample number of common points for compiling partial surveys of the area, while its uniform precision permits of adjusting, without question, the partial surveys or revision photographs to the base map. Heretofore, it has usually been less expensive to make a complete re-survey rather than attempt to reconcile the discrepancies of the former survey with the new data. This uniformity of precision in mapping, backed by the permanent record of the photographs, is a second great contribution of aerial photography to surveying.

Regarding the question as to which method or machine for stereo-photographic mapping is best suited to a given problem, the art is being developed too rapidly and is too much in the controversial stage to permit of positive statements at this time. Aside from the process developed by Arthur and Norman Brock and their associates, and the aerocartograph, which have been successfully used in the United States, it has been reported that the stereo-planograph (Fig. 6) has been adapted to plotting quadruple-lens photographs having a field of 83° without resetting. Such a wide field might well increase accuracy and reduce costs, if successful, and deserves the consideration of any one actively interested. The British have developed a stereogoniometer for determining the successive orientation of the photographs to each other along a flight strip, the map relations being determined later by calculation. This method would seem to have practical advantages for extensive mapping on scales smaller than 1:10 000, and has been described by Captain M. Hotine[11] who also reviews other machines and methods critically. For accessible country, apparently it has been found satisfactory and somewhat less costly in some cases[12] to take planimetry only from the aerial photographs and to contour the map sheets on the plane-table. This method is likely to be used frequently on account of the high first cost of the machines. Since the various stereo-plotting machines cost from $15 000 to $30 000 to construct, or to import into this country, and also require specially trained operators, considerable mapping is required to underwrite their use. Present economic conditions have undoubtedly retarded the development and use of such machines considerably.

Certain inaccessible areas of considerable relief, such as the back canyons of Zion National Park, or the wooded mountains of the Olympic Peninsula, cannot be adequately mapped, except at prohibitive costs, unless the aerocartograph or some other method of stereo-photographic mapping be used. For mapping terrain of considerable relief on scales from 1:3 000 to 1:10 000,

[11] "Surveying from Air Photographs," by Capt. M. Hotine, R. E.

[12] "Developing the Upper Mississippi—Surveying and Mapping Methods," by L. Ylvisaker, *Civil Engineering*, Vol. 1, No. 15, pp. 1357–1360.

undoubtedly stereo-photographic mapping methods give superior accuracy at lower costs. It would be appreciated, if, in closing the discussion, Colonel Birdseye would give some comparison of the relative costs and technical advantages of mapping various types of terrain at different scales by means of the aerocartograph, the photo-plane-table method, and the plane-table alone. In making this request, the writer is aware that it is almost impossible to discuss costs intelligently without reference to particular localities and map specifications, but Colonel Birdseye is believed to be better qualified than any one known to the writer to define the present economic and technical limits of the different methods.

LOWELL O. STEWART,[13] ASSOC. M. AM. SOC. C. E. (by letter).[13a]—Machines that are used to make maps from aerial photographs are described in an interesting and lucid manner in this paper. The author's statement, "that a map properly made by adequate photographic surveying methods must be more consistent throughout than one made by what may be called the old-fashioned ground-survey method," is a point that deserves emphasis.

The question of relative accuracy always enters the discussion when the merits of various topographic methods and instruments are being considered. In that connection the surveyor should keep in mind that a map is a representation of the original to a certain scale and contour interval, and must be a compromise between absolute accuracy and time and cost. Certain features must be omitted. Others that do appear on the map are designated by conventional symbols. Therefore, in judging the quality of any map the scale, the cost, and the time required to make it, must be considered. High standards of precision in the detailed field work cannot be written into the finished map, because of plotting limitations. Probably the limit of precision in scaling lies between 0.01 in. and 0.02 in., and ordinary protractors are good only to the nearest 10′ or 15′.

A matter of greater importance has to do with the detailed field work. Any one who has taken "shots" on the ground for contours knows how important a part judgment plays in the selection of those points. Later, after the points have been plotted, the contour lines must be interpolated. This requires more or less generalization, the amount depending upon the minuteness with which the ground was covered by "shots." Therein lies an important factor in determining the accuracy of the finished map. In rugged country the lines may be very close to their proper positions, while on easy slopes they may be far from that ideal.

Engineers are prone to accept locations as correct on a map in spite of the fact that such maps differ widely from one another in accuracy. It is time that surveyors develop standards for checking topographic maps, and apply them properly. Some progress has been made along that line on extensive city surveys and on the recent surveys of the Upper Mississippi River. Two tests have been applied. The first, for position of general features, usually requires that all features shall be located with such accuracy

[13] Asst. Prof., Civ. Eng., Iowa State Coll., Ames, Iowa.

[13a] Received by the Secretary February 23, 1932.

that no error is measurable at the map scale, which means that a given feature must be within 0.01 to 0.02 in. from its correct position on the map; the second, a test for contour accuracy, specifies that none (sometimes 10%) of the test points may be in error on the map more than one-half a contour interval. The latter test seems insufficient in flat country.

Likewise, a sound field technique for testing should be developed after the standards of accuracy for the finished map have been set up. Different types of terrain would not require the same detailed field procedure; but surveyors should have those requirements and differences so clearly set forth that any one could test a map and grade it as to its acceptability according to the recognized standards. When that is done, and the requirement for testing is included in the specifications for every topographic mapping project, the pseudo-maps that are now frequently made, will disappear. Accurate maps will cost more than hurried and inaccurate maps; but the assurance of their reliability is worth much more than the added cost.

These standard specifications should have an important bearing on the development of serial mapping. After all, a map is a map until it has been proved otherwise. The advocates of aerial methods may state, with justification, that better and more consistent maps may be made from aerial photographs than from ground surveys alone. The facts may be learned through experience with maps that have been made by the two methods and used for identical purposes; but a better and more scientific way seems to be through the use of standard tests.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FULTON STREET, EAST RIVER TUNNELS, NEW YORK, N. Y.

Discussion

By Messrs. H. J. King, S. M. Swaab, and Jacob Feld

H. J. King,[4] M. Am. Soc. C. E. (by letter).[4a]—While comparatively few engineers are faced, during their professional careers, with the problem of driving tunnels under such difficult conditions as those described by Mr. Killmer, nevertheless this extremely valuable contribution can be studied with great profit by tunnel men engaged in work in normal air.

The writer is acquainted with several tunnels driven under more or less populous centers over which subsequent subsidence of the ground surface has been the basis of actions brought to recover losses for heavy damages to adjacent buildings and other improvements. In certain types of ground this subsidence may not be evident for several years after the completion and acceptance of the tunnel by the owner.

In the writer's opinion the application of the apparatus for ejecting pea gravel, so clearly and interestingly described by Mr. Killmer, could well be extended to include many liner tunnels driven under areas where surface subsidence is objectionable. In many cases the gravel treatment alone would be entirely sufficient to prevent subsidence, and the necessity for the more costly grouting treatment could be obviated.

S. M. Swaab,[5] M. Am. Soc. C. E. (by letter).[5a]—The work done by Mr. Killmer on the Fulton Street Tunnels in New York City represents the highest development of the art of subaqueous tunneling. While shield tunneling is an English invention and the first shield-driven tunnels were built in England, still the art has received its maximum development in methods, safety,

Note.—The paper by Miles I. Killmer, M. Am. Soc. C. E., was presented at the meeting of Construction Division, New York, N. Y., January 16, 1930, and published in December, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[4] Supt., A. Guthrie & Co., Inc., St. Paul, Minn.

[4a] Received by the Secretary December 28, 1931.

[5] Cons. Engr., Philadelphia, Pa.

[5a] Received by the Secretary February 24, 1932.

and economy, in the region of the Port of New York since the date of the building of the Hudson and Manhattan Tunnels by the late C. M. Jacobs and J. V. Davies, Members, Am. Soc. C. E.

On each of the several tunnels as it was built, certain innovations were made until the process of building subaqueous tunnels can now be said to be fairly well established, the size of the tunnel and the nature of the ground being the same.

On the Fulton Street Tunnels there has been made the most consistent, and, at the same time, the greatest and best progress (except that made on those tunnels where the material of the face practically flowed into the tunnel through the openings in the shield, or where the shield merely displaced the material) on any of the subaqueous tunnels in the neighborhood of New York; and for workmanship, alignment, and grade, they have not been surpassed.

The specifications issued by the Board of Transportation for this work left to the contractor the choice of electric or steam power for the operation of the compressor plants, and the contractor chose to use electric power. This was made possible by improvements in compressor machinery. The plate valves used to-day are lighter in weight and, at the same time, provide greater area for the air to pass through and cause less restriction than the poppet-valves formerly used. Consequently, the machines can be operated at much higher speeds than formerly, being, in fact, direct-connected to the motors and operating at full motor speed without restriction. However, in adopting electric power, one has to be alert because of accidents to the feeder lines to which there is always some liability (such as that to which Mr. Killmer calls attention), even if the power lines come from different sources. Proper protection of the circuits by lightning arrestors can readily be provided at comparatively little expense. The expense of supplying this protection is, in fact, so small that one cannot readily afford to be without it.

The automatic valves for controlling the output of air from the low-pressure compressors were, undoubtedly, designed to be operated by moderately high, low-pressure air. Consequently, when the low-pressure air was reduced materially by a blow-out in one of the headings, there was not sufficient pressure available to operate the valves. Mr. Killmer immediately overcame this by cutting into the line of the old compressor with hand-operated valves, and the pressure was thus boosted to provide for the emergency. Subsequently, a recurrence of this condition was prevented by taking the air supply for valve operation from a receiver connected with the high-pressure machines.

The plant which is described in detail in Mr. Killmer's paper was ample for the purpose. Three of the low-pressure compressors would have been sufficient, at times, on the Manhattan side, and six might have been used on the Brooklyn side to advantage, owing to the fact that the material on the Manhattan side held the air better than that on the Brooklyn side of the river.

The connection between the high-pressure and the low-pressure air lines is very advantageous and, as Mr. Killmer states, was availed of several times in connection with this work.

The shields which were used on Fulton Street fulfilled every purpose and stood up so well that in the subsequent work done by Mr. Killmer on Rutgers Street, this same shield design was used without any substantial change.

The innovation made in the method of tunneling under atmospheric air in the station on High Street was, undoubtedly, a very desirable one, since there has been no material left in the ground which, in the nature of things, will deteriorate.

The method of tunneling in soft ground without air (previously used on the stations built in connection with the several subaqueous tunnels, prior to the building of the Fulton Street Tunnels), permitted a large quantity of heavy timber to remain in the ground, but in the substitute method used by Mr. Killmer, this is not so. The advantages of this method over that formerly used on similar work are readily recognizable and have been highly satisfactory. In any event, it is quite possible that there will be less disturbance of the utility structures, buildings, and street paving in the future than has occurred in connection with any of the other work heretofore built.

In Fulton Street, Manhattan (which is about 55 ft. in width between the houses), the tunnels were driven under compressed air through sand below the level of the ground-water, in many instances below, and in all cases extremely close by, the foundations of the buildings, without causing injury worthy of the name, and, in fact, without appreciable settlement. This was done in advance of the building of the station between Broadway and William Street. The latter work was done from the surface after the tunnels had been driven. It was originally intended to underpin the buildings on both sides of the street in advance of the driving of the tunnels. This was not done, however; but for the purpose of protecting and securing the buildings during and after the construction of the station, permanent bulkheads, built piecemeal, in isolated sections, in individual pits, were constructed in front of them on either side of the street and just beyond the station walls, in advance of the building of the station. These bulkheads, when completed, were solidly braced across the cut provided for the station, as the excavation was made, and the station itself was subsequently built within the lines of the bulkheads thus provided.

The work on the station in Fulton Street, Manhattan, which was one of the most difficult operations to be performed in connection with this contract, was distinctly illustrative of what could be done by way of tunneling in close proximity to high and heavy buildings. After the completion of the tunnels and immediately over them, the earth prism between the surface and the top of the tunnels was removed in a bulkhead-protected trench, without permanent injury to any of the buildings, and with a minimum of disturbance during their occupancy. Incidentally, there was as little or possibly less settlement of the individual buildings than would have occurred had they been previously underpinned.

The method of following up the erection of the iron close by with pea gravel and of subsequently injecting into this cement grout had the desired effect of preventing surface settlements to the largest extent that they have

ever been prevented in any work of this character in New York City, or elsewhere.

Indeed, it would be difficult, to say the least, for one to conjecture what might possibly happen on work of this character. One must always be on the alert. For the ordinary things that happen, a man may be, and is to some extent, prepared. It requires an ingenious, resourceful, and quick-witted man, however, to be prepared to meet unexpected contingencies as they arise, and then there must be a liberal allowance of "good luck," or its equivalent, no matter by what name it is called. In addition, a good organization properly directed, adequate financial resources, and a properly designed plant, such as was here available, are required to complete a contract, such as this, successfully.

Mr. Killmer is to be congratulated for the very ample and lucid description of the plant which he has given the Society in this paper, and the Society should, in turn, be glad to have this description of one of the most important pieces of work conducted in New York City in recent years, among the similar papers descriptive of this character of work carried out by its members. It is needless to say that it was a pleasure to be associated with Mr. Killmer on this work.

Jacob Feld,[6] Assoc. M. Am. Soc. C. E. (by letter).[6a]—The author has omitted mention of the methods for checking line and grade in the two sections of each river tunnel by which such remarkable coincidence as that shown in Fig. 16, was obtained. The writer has given line and grade in rock and earth tunnel construction under structures, and appreciates how much detailed work is required in providing accurate control points in blind headings to permit accurate meeting of such headings. The same operation must be considerably more difficult in tunnels under water, because of the longer sections and because of the movements which must occur in each section due to such conditions as change in tide levels and possibly motion of the soil surrounding the tunnel shield.

Mr. Killmer is to be congratulated on his complete and clear description of the construction of the tunnel under water, probably one of the best descriptions that has ever been published.

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[6] Cons. Engr., New York, N. Y.

[6a] Received by the Secretary February 4, 1932.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his **eligibility.** It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that errors in the record be pointed out and a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from April 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

LIST OF APPLICANTS.

Names and Addresses of Applicants for Admission and for Transfer on this List.
Arranged Alphabetically.

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FOX, FREDERICK J.	Bronx, N. Y.	80	SHUBIN, SYDNEY A.	Pittsburgh, Pa.	90
FRANKLIN, WILLIAM R.	Brownsville, Tex.	94	SMITH, CHESTER W.	Huntington, W. Va.	91
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GOODRIDGE, RICHARD S.	Los Angeles, Cal.	80	SONNE, JULIUS A.	Burlingame, Cal.	87
GREER, ROBERT C., JR.	Springfield, Pa.	80	SOULE, JOHN E.	Pensacola, Fla.	95
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HIPP, CHARLES L.	Kansas City, Mo.	81	THOMAS, WALLACE A.	Ferguson, Mo.	96
HIPP, THOMAS M.	Whittenburg, Tex.	81	THOMPSON, THORALF S.	Brooklyn, N. Y.	96
HUBKA, VENCEL A.	Traverse City, Mich.	81	WILLIFORD, CARL L.	Dallas, Tex.	92
JOHNSON, IRVING L.	Sacramento, Cal.	81	WILSON, PERCY S.	Glen Ridge, N. J.	93
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The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

The number in the center above each record indicates the serial number of the applicant for the current year, and that at the left the district in which he resides.

The abbreviations in Italics represent respectively, *TT*, Total Time; *SP*, Sub-Professional Work; *P*, Professional Work; *RC*, Responsible Charge; *D*, Design. The figure for Total Time is determined by adding one-half the time spent in Sub-Professional Work to the time spent in Professional Work. The figures showing the amount of time spent in Responsible Charge and on Design are the estimate of the Applicant. The allowance of four years for graduation or of one-half of a year for each academic year successfully completed in an engineering college without graduation is included in Total Time and Professional Work.

FOR ADMISSION

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(1) **ALBRECHT, ERICH WOLFGANG**, 66 Park Ave., Babylon, N. Y. (Age 49. Born San Francisco, Cal.) 1910 Geographical Engr., Acad. of Tech., Charlotteburg, Berlin, Germany. July 1902 to April 1903 Chainman and Rodman, coal mine surveying, Dunkville Mining Syndicate, St. Louis, Mo., and May 1903 to Sept. 1904 Chainman, Rodman and Instrumentman, Okaw & Eastern R. R. *TT 1: SP 1.*—Oct. 1904 to Oct. 1910 student.—Oct. 1910 to Oct. 1911 Computer on surveys and subdivisions, Lawyers Title Insurance Co., New York City. *TT 0.5: SP 0.5.*—Nov. 1911 to Oct. 1914 with Dominion Civil Service as Instrumentman and Acting Res. Engr., on location and construction, National Transcontinental Ry., and (2 years) Jun. Asst. Engr., Astronomical Branch, triangulation and astronomical operation, International Boundary surveys, Lake of the Woods region. *TT 1.5: SP 1.5.*—Nov. 1914 to Oct. 1918 giving courses in Civil Engineering and Surveying at Eng. School, and private practice, surveying, designing water-supply systems and sewerage projects. Tacoma, Wash., and (after April 1917) investigating irrigation projects, Pacific Watershed, Northern Mesa (Dist. del Norte, Lower California), highway location and construction, mining and preliminary railroad surveys, Ensenada, Lower California, Mexico. *TT 3.8: P 3.8: RC 3.8.*—Nov. 1918 to July 1920 Engr. in Chg., Esperanza Mining Co., El Oro, Estado, Mexico, directing topographical operations and triangulation. *TT 1.7: P 1.7: RC 1.7.*—July 1920 to March 1927 with Mexican Federal Govt. Service, until Feb. 1925 with Dept. of Agriculture, as First Astronomer, determining astronomically geographic coordinates on boundary surveys, conducting expeditions, Geodetic Astronomer, geodetic and astronomical operation, National Observatory, Tacubaya, Mexico City, First Engr., Comm. National Agraria, geodetic operation in Yucatan, conversion of geodetic data, then Research Eng. Specialist conducting topo-photographic experiments and studies, etc. Dept. of Finance, Mexico City, and after March 1926 First Topographer, investigating Yagin River irrigation project in Chihuahua and hydro-electric and irrigation project in Michoacan, selecting dam sites, National Irrigation Comm. (Dept. of Agricultura y Fomento), Mexico City. *TT 6.5: P 6.5: RC 6.5.*—May-Dec. 1928 Geographical Engr., D'Arcy Exploration Co., Ltd., Medellin-Bogota, Colombia, determining astronomically geographic coordinates of control points for aero-photographic surveys, Anglo-Persian Oil Co. *TT 0.6: P 0.6: RC 0.6.*—May to Sept. 1929 Engr. in Chg. of highway location and construction, Putnam County Planning and Developing Comm., Carmel, N. Y. *TT 0.2: SP 0.1: P 0.1: RC 0.1.*—Oct. 1929 to Jan. 1930 Instrumentman topographical mapping and field work, Port of New York Authority, Ft. Washington, approaches of Washington (Hudson River) Bridge, New York City. *TT 0.1: SP 0.1.*—Jan. to June 1930 Topographical Draftsman, making topographical map of stone quarries, New York Trap Rock Corporation, Newburgh, N. Y. *TT 0.2: SP 0.2.*—July-Sept. 1930 Inspector, highway

construction, in charge of construction of highway (2½ miles), including two bridges, Somers, N. Y., Westchester County Highway Comm. *TT 0.3: P 0.3: RC 0.3.*—Oct. 1930 to July 1931 Research Eng. Specialist, U. S. Engr. Corps, Long Branch, N. J., Observing Station, Oceanographic studies, conducting experiments for effects of wave action on shore structures, sand movement and beach erosion, calculation of results, reports. *TT 0.6: SP 0.2: P 0.4: RC 0.4.*—Jan. 1931 to March 1932 Topographical Draftsman and Computer, Grade 5, Long Island State Park Comm., Babylon, N. Y., calculating, computing right-of-way takings and stakeout maps, determining and calculating astronomical meridians, with high precision, geodetic computations. *TT 0.7: SP 0.6: P 0.1: RC 0.1.*—*TT 17.7: SP 4.2: P 13.5: RC 13.5.* Refers to C. E. Beam, A. H. Beyer, W. Bowie, V. Gelineau, G. H. Matthes, E. K. Smoot.

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(9) **AREN, ARMIN CONRAD HENRY,** 3452 Harlem Pl., Cincinnati, Ohio. (Age 43. Born Bellevue, Ky.) 1912 B. C. E., Univ. of Cin. *TT 4: P 4.*—June 1909 to Sept. 1910 Draftsman, Cincinnati, Hamilton & Dayton Ry. *TT 0.6: SP 0.6.*—March 1916 to Feb. 1918 Chemist, Glass Factory.—Feb. to Dec. 1918 with C. W. S., U. S. Army.—June 1912 to March 1916 and Feb. 1919 to date with Structural Dept., Div. of Highways, City of Cincinnati, Ohio, until May 1913 as Transitman, then Draftsman, detailing and since Feb. 1919 Asst. Structural Engr., 3½ years designing bridges, viaducts, walls, steps, etc., and since Aug. 1923 Head of Structural Group, supervising studies, designs, working drawings, details, estimates and specifications for building and repairing of viaducts, bridges, trestles, culverts, drains, retaining walls and steps (concrete, reinforced concrete, steel, timber and stone masonry), supervises contract contruction and acts as Consultant on work done by City forces; makes studies and general designs for grade-crossing elimination, assists in drafting ordinances, etc. *TT 14: SP 1.9: P 12.1: RC 8.6: D 12.1.*—*TT 18.6: SP 2.5: P 16.1: RC 8.6: D 12.1.* Refers to W. W. Carlton, H. H. Kranz, C. N. Miller, F. L. Raschig, J. E. Root, H. F Shipley.

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(7) **BEALL, MARSH FLAGG,** 934 Olivia Ave., Ann Arbor, Mich. (Age 22. Born Canton, Ohio.) 1932 B. S. in C. E. Univ. of Mich. *TT 4: P 4.*—Summers 1924–1930 Draftsman, Michigan Bell Telephone Co., Grand Rapids, Mich.—*TT 4: P 4.* Refers to L. W. Goddard, W. C. Sadler, R. H. Sherlock, C. O. Wisler, J. S. Worley.

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(1) **BECKWITH, WILLIAM PERCY,** 22 Sackville Road, Vineyard Pen, Kingston, Jamaica. (Age 41. Born Kingston-upon-Hull, Yorkshire, England.) 1904 to 1907 student in Elec. Eng., Kingston-upon-Hull, Yorkshire (England) Tech. Coll.—1907 to 1909 successively with Eng. Staff, National Telephone Co., Hull and North Eastern (London & North Eastern) Ry. *TT 1.1: SP 1.1.*—1909 to 1910 with Humber Elec. Co., Hull. *TT 0.4: SP 0.4.*—Oct. 1910 to April 1921 with Corps of Royal Engrs., until June 1912 in military and electrical engineering training, qualifying as Mil. Electrician, June to Dec. 1912 at Pembroke Dock, South Wales, in charge of erecting searchlight and engine at Thorn Island, Milford Haven, Dec. 1912 to July 1914 at Jamaica installing electric lighting and fixing up power station at Port Royal, July 1914 to Nov. 1918 in charge of group of searchlights, engines, etc. at an outstation, Nov. 1918 to June 1920 Station Engr. in charge of power station, Port Royal, and after July 1920 Acting Clerk of Works at Guernsey, Channel Islands, on maintenance of roads, bridges, etc., also designed, estimated and erected twelve sanitary annexes to buildings and supervised erection of pavilion and stand on sports ground. *TT 9: P 9: RC 9: D 0.2.*—April 1921 to Oct. 1922 student, School of Military Eng. at Chatham, Kent.—Dec. 1922 to July 1931 Clerk of Works, H. M. War Dept., until August 1923 at Bulford, Salisbury Plain, Wiltshire, on maintenance of hutted camps, Aug. 1923 to March 1924 at Weedon, Northants, in charge of Ordnance Dept., on design and estimates for quarters, sanitary annexes to quarters (2-story building), dwelling houses, etc., supervised erection of a shop, reconstruction of a mess and a dwelling house, etc., March 1924 to March 1925 at Sheerness, Kent, in charge of seawalls, groynes, ordnance depot and fortifications, on Isles of Sheppey and Grain; including underpinning of 30-ft. tower, preventing collapse of about 100 ft. of seawall, Isle of Grain, removal and re-erection of 100-ft. steel lattice tower on seawall, Sheerness; erection of bridge for removal of 100-ton guns on Isle of Grain; March 1925 to July 1931 in Jamaica, on design and estimate for barracks, stables, quarters, hospital construction and reconstruction and after Dec. 1925 Engr. in charge of roads, drainage, water supply, cemeteries, rifle ranges, etc.; after 1926 also responsible for design, layout and estimate for complete sports ground (approx. $25 000), design and estimate for headquarters (reinforced concrete, approx. $25 000), design and construction of

reinforced gas chamber, oil and petrol store, retaining wall, reinforced concrete spectators stand on sports ground, flour store for Garrison Bakery, office, swimming bath and extension to officers' club, construction of reinforced concrete band stand, completion of Regimental Inst., consisting of design and construction of basement storage for wines, foodstuffs, etc., design and construction of block of offices, plan, section of levels, specification, contract documents for 9-in. G. S. W. sewer, supervising work; estimate for conveying waste water from swimming baths, septic tank installation for an isolated house, design, estimate and brief specification for guard room (reinforced concrete), design, estimate and erection of motor roller shed, including fitters' workshop and store (reinforced concrete), also sat as technical member on two boards; June 1930 passed War Office technical examination. *TT 8.5: P 8.5: RC 8.5: D 8.5.*—July 1931 to date Overseer of Works, Gen. Penitentiary, Kingston, superintending Productive and Mech. Depts., repairs and alterations to buildings, measuring, estimating and recording value of prison labor; supervises and is responsible for brickyard and quarry and for ballasting of ships, has charge of fire appliances, examines prison boats, inspects prison for necessary repairs, etc. *TT 0.8: P 0.8: RC 0.8.*—*TT 19.8: SP 1.5: P 18.3: RC 18.3: D 8.7.*—Refers to E. N. Bancroft, F. L. Bronstorph, H. J. Dignum, S. C. Henriques, A. A. Simms.

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(2) **BRADFORD, WILLIAM HARLOW,** R. F. D. No. 7, Box 17, Augusta, Me. (Age 25. Born Augusta, Me.) 1927 graduated in Arch. Constr., Wentworth Inst.—June 1927 to date with Maine Highway Comm., until Dec. 1928 as Bridge Inspector on construction, surveys and drawing and designing small bridges, then Bridge Draftsman, designing reinforced concrete bridges, estimating cost of bridges, making final estimates of quantities, some steel construction, and since Jan. 1932 Jun. Engr. on same work. *TT 4: SP 0.7: P 3.3.* Refers to C. W. Banks, H. L. Doten, L. N. Edwards, R. D. Field, P. H. Glover, E. D. Kingman.

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(3) **BROWN, FRANCIS LEO,** 10 Lafayette St., Whitehall, N. Y. (Ave 22. Born Albany, N. Y.) 1931 Chem. Engr., Rens. Pol. Inst. *TT 4: P 4.*—Summer 1931 Eng. Asst., New York State Dept. of Highways.—Summers 1929 and 1930 Rodman and Student Surveyor, and Nov. 1931 to date Chainman, Delaware & Hudson R. R. *TT 0.2: SP 0.2.*—*TT 4.2: SP 0.2: P 4.* Refers to L. W. Clark, T. R. Lawson.

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(5) **BROWNE, FREDERICK LEE,** 9709 Warren St., Forest Glen, Md. (Age 24. Born Washington, D. C.) 1932 B. S. in C. E., Purdue Univ. *TT 4: P 4.* Refers to C. B. Andrews, E. C. Bebb, W. K. Hatt, S. C. Hollister, E. C. Webster.

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(15) **CALDWELL, DAVID KING,** P. O. Box 151, Tyler, Tex. (Age 44. Born Scottsboro, Ala.) 1908 B. S. in C. E., and 1912 C. E., Ala. Pol. Inst. *TT 4: P 4.*—1908 to 1909 Inspector and Instrumentman on highway work in Montgomery, Ala. *TT 1.2: SP 0.2: P 1.*—1909 to 1910 Asst. Engr., City of Montgomery, laying out and superintending construction of water-main extensions and pumping plant. *TT 1: P 1.*—1910 to 1911 Draftsman, U. S. Engr. Office, Montgomery, on topographic maps and construction of dam and locks on Coosa River near Rome, Ga. *TT 0.8: P 0.8.*—1911 County Engr., Andalousa, Ala., on surveys and plans for road system. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—1911 to 1913 County Engr., Elmore County, Ala., and Res. Engr., State Highway Dept., designing and constructing roads, including state highways in Elmore County. *TT 2: P 2: RC 2: D 0.5.*—1913 to 1914 Asst. to Parish Engr., and 1916 to 1918 Parish Engr., Caddo Parish, La., being Inspector in charge of road and bridge projects, and (after 1916) constructing extensions to County and State road system and in charge of maintenance of roads and bridges. *TT 3.3: P 3.3: RC 2.8: D 0.7.*—1914 to 1916 County Engr., Gregg County, Tex., constructing county roads, including State highways and bridges across Neches River. *TT 2.1: P 2.1: RC 2.1: D 0.5.*—1918 to 1919 Highway Engr. and Sr. Highway Engr., U. S. Bureau of Public Roads, 4 months on forest road work in Florida, 2 months on Federal inspection work in Alabama and Tennessee, and 8 months in charge of Federal Aid work in Mississippi. *TT 1.2: P 1.2: RC 1.2: D 0.2.*—1919 to 1924 County Engr., Smith, County, constructing state higways and maintaining state and county roads. *TT 4.3: P 4.3: RC 4.3: D 1.*—1924 to 1931 principally on highway and bridge construction and city paving, including state highway construction in Smith, Bowie, Walker, San Jacinto and Panola Counties and city paving in Tyler, Carthage Timpson and Huntsville. *TT 6: P 6: RC 6: D 2.*—1931 to date Res. Engr., Texas State Highway Dept.; County Engr., Houston County; Engr. in charge of Construction and of plans on state highway

projects in Houston, Walker, Trinity and Anderson Counties. *TT 1.3: P 1.3: RC 1.3: D 0.3.*—*TT 27.5: SP 0.2: P 27.3: RC 20: D 5.6* Refers to J. T. Bullen, J. M. Garrett, G. Gilchrist, T. E. Huffman, C. H. Kendall, J. T. L. McNew, J. M. Page.

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(1) **CONNOR, JOHN FRANKLIN,** 87 Grant St., Tompkinsville, N. Y. (Age 26. Born Williamsbridge, N. Y.) 1932 B. S. and C. E., Col. of City of N. Y. *TT 4: P 4.*—Feb. 1932 to date Eng. Asst., Board of Transportation, City of New York, acting as Rodman, Chainman, Estimator and Calculator. *TT 0.1: SP 0.1.*—*TT 4.1: SP 0.1: P 4.* Refers to R. E. Goodwin, F. O. X. McLoughlin.

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(11) **CORNICK, FREDERICK JOSEPH,** 344 South Santa Anita Ave., Pasadena, Cal. (Age 42. Born Detroit, Mich.) Student, Univ. of Ariz. (1912 to 1913 and 1916 to 1917) and Univ. of Cal. (1925).—Sept. 1917 to Feb. 1919 Asst. Engr., Lindsay-Strathmore Irrigation Dist., Lindsay, Cal., on hydrographic investigation of surface and ground water supply and demand, duty of water for irrigation, water rights litigation. *TT 1.3: P 1.3: RC 0.5.*—March to May 1919 Levelman, U. S. Dept. of Public Roads, Pasadena. *TT 0.1: SP 0.1.*—Oct. 1919 to Jan. 1920 Inspector, City of Phoenix, Ariz., on street improvement and construction of drainage system. *TT 0.2: SP 0.2.*—Jan. 1920 to May 1921 Hydrographical Engr. with Arizona Water Commr., Phoenix, in charge of water rights investigations on upper Gila River. *TT 1.3: P 1.3: RC 1.3.*—March to Oct. 1922 Field Engr. with A. L. Sonderegger, Cons. Engr., Los Angeles, Cal., subdivision and hydrographic surveys. *TT 0.6: P 0.6: RC 0.6.*—Nov. 1922 to March 1923 Hydrographer, Vail Ranch Co., Temecula, Cal. and O'Neil Ranch Co., Oceanside, Cal., hydrographic investigation on Temecula Creek and Santa Margarita River. *TT 0.3: P 0.3: RC 0.3.*—June to Dec. 1923 Draftsman, U. S. Indian Irrigation Service, Los Angeles, Cal. *TT 0.3: SP 0.3.*—Dec. 1923 to June 1925 Hydr. Engr., State Div. of Water Rights, in charge of water-rights and hydrographic investigations, stream runoff determinations, ground water studies at San Dimas, Cal. *TT 1.5: P 1.5: RC 1.5.*—Aug. 1925 to April 1926 Chf. of Party, Victor Girard Co., Girard, Cal., on subdivision, road location and improvement surveys. *TT 0.7: P 0.7: RC 0.7.*—April to Sept. 1926 Hydr. Engr. and Water Master, and Nov. 1926 to April 1927 Hydr. Engr., Div. of Water Rights, State of California, being Water Master in charge of distribution of irrigation water, Cedarville, Cal. and after Nov. 1926 on hydrographic investigations, stream gauging, ground-water investigations in Los Angeles County. *TT 0.8: P 0.8: RC 0.4.*—April 1927 to date Asst. Engr. and Hydrographer, Los Angeles County Flood-Control Dist., being Asst. to Chf. Hydrographer, on hydraulic studies, hydrographic investigation, operation and maintenance of twelve flood-control dams. *TT 4.9: P 4.9: RC 4.9: D 3.*— *TT 12: SP 0.6: P 11.4: RC 10.2: D 3.* Refers to H. Conkling, E. C. Eaton, H. Forbes, H. E. Hedger, L. C. Hill, E. Hyatt, A. L. Sonderegger.

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(1) **CRABTREE, FREDERICK HOWARD,** 180 Ashland Ave., Bloomfield, N. J. (Age 31. Born Boston, Mass.) 1923 B. S. in Civ. Eng., Tufts Coll. *TT 4: P 4*—July 1923 to Nov. 1925 and Sept. 1926 to July 1930 with Stone & Webster Eng. Corporation, until Nov. 1925 as Instrumentman and Field Engr. on construction of dam and hydro station at Iron Mountain, Mich., for Ford Motor Co., relocation and construction of highways, railroads and bridges for Bartlett's Ferry Dam above Columbus, Ga., etc., and (April to Dec. 1924) Instrumentman, line and grade work, figuring quantities, etc. on hydro and steam stations, dock and tunnels for Ford Motor Co. at St. Paul, Minn.; Sept. 1926 to May 1929 Field Engr. on construction of Cathedral of Learning, Univ. of Pittsburgh, line and grades, concrete and steel inspection and after May 1929 Res. Engr. on construction of laboratory and industrial buildings for Westinghouse Lamp Co., Bloomfield, N. J. *TT 6.1: P 6.1: RC 1.2.*—Nov. 1925 to Sept. 1926 subdivision, surveys and some highway and sewer work in Florida. *TT 0.4: SP 0.4.*—July 1930 to Jan. 1932 Res. Engr., A. L. Hartridge Co., in charge of instrumentmen and inspectors and coordinating subcontractors on construction of 25-story office building for Stone & Webster, Inc., in New York City. *TT 1.5: P 1.5: RC 1.5.*—*TT 12: SP 0.4: P 11.6: RC 2.7.* Refers to C. L. Bell, E. F. Blakeslee, J. E. Bomar, H. P. Burden, C. E. Nichols.

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(1) **DOOLITTLE, FRANK BALDWIN,** 1845 Grand Central Terminal, New York City. (Age 39. Born Mt. Vernon, N. Y.) Prof. Engr., New York State. 1913 Ph.B. in Civ. Eng., Sheffield Sci. School, Yale Univ. *TT 3: P 3.*—June to Sept. 1913 Structural Steel Detailer with Post & McCord, New York City, on construction. *TT 0.1: SP 0.1.*—Sept.

1913 to May 1914 Draftsman and Designer, Titchener Iron Works, Binghamton, N. Y., on light structural and ornamental work. *TT 0.4: SP 0.4.*—May to Sept. 1914 Draftsman with Gibbs & Hill, New York City, on Pennsylvania R. R. electrification, catenary system. *TT 0.2: SP 0.2.*—Oct. 1916 to April 1917 Estimator, Austin Co., Bridgeport, Conn., on building construction. *TT 0.2: SP 0.2.*—Sept. 1914 to Oct. 1916 and April 1917 to May 1931 with Elec. Div., M. of W. Dept., New York Central R. R., until Oct. 1916 as Rodman, Transitman and (4 months) Asst. Supervisor of Track, April 1917 to Feb. 1924 Asst. Engr. on surveys, plans for minor structures and special investigation (about 3/4 year), Bridge Inspector and supervising some maintenance repair work (3 4/5 years) and Gen. Foreman of bridge maintenance, supervising timber, steel and mason gangs on bridge repair and erection (2 2/5 years); Feb. 1924 to May 1927 Asst. Div. Engr., assisting in charge of maintenance of Division and of Engr. Corp.; May to June 1927 Supervisor of Track, in charge of maintenance, main line; after June 1927 Supervisor of Structures, in charge of maintenance of bridges and buildings, including drawbridges, turntables, engine houses, coaling plants, and in responsible charge of bridge erection and removal, falsework under tracks, etc. *TT 14.7: SP 1.3: P 13.4: RC 4.*—May 1931 to date New York Representative, Leake & Nelson Co., Bridgeport, Conn., investigating steel structures, especially bridges, and recommending repair by electric arc welding, including sales work, handling contracts and directing work. *TT 0.8: P 0.8: RC 0.8.*—*TT 19.4: SP 2.2: P 17.2: RC 4.8.* Refers to A. W. Carpenter, C. L. Dayton, W. N. Hazen, Z. H. Sikes, C. H. Sorensen, W. L. Unger, H. T. Welty.

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(12) DUNKLE, DUDLEY ANDREW, 459 East Fifteenth St., Tacoma, Wash. (Age 34. Born Beaver Falls, Pa.) 1920 B. S., Washington & Jefferson Coll.—July to Dec. 1918 Second Lieut., Field Artillery, C. O. T. S., U. S. Army, Camp Taylor, Ky.—July 1920 to May 1923 Traffic Mgr., Chesapeake & Potomac Telephone Co.—March to Oct. 1924 Shipping Clerk, Laborer in foundry, Snow Mfg. Co., Los Angeles, Cal.—June 1923 to Feb. 1924 and Nov. 1924 to July 1929 with Western Concrete Pipe Co. (American Concrete Pipe Co.), until Aug. 1923 as Laborer and Timekeeper, making concrete pipe, and after Sept. 1923 Constr. Supt. installing concrete water and sewer systems in Southern California, etc. *TT 5.3: P 5.3: RC 5.3.*—Aug. 1929 to date Vice-Pres. and Gen. Mgr., American Concrete Pipe Co. of Washington, manufacturing and installing water-supply and sewer systems. *TT 2.7: P 2.7: RC 2.7.*—*TT 8: P 8: RC 8.* Refers to C. D. Forsbeck, H. M. Hadley, C. W. Kief, C. N. Reitze, W. A. Whiting, N. D. Whitman.

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(1) EAPEN, MATHEW, Y. M. C. A., Esplanade, Madras, South India. (Age 41. Born Chengannur, Travancore State, South India.) April 1915 to Oct. 1917 student, Civ. Eng. Branch, Univ. of Edinborough, Scotland. *TT 1: P 1.*—1917 to 1922 not on engineering work.—Jan. to Aug. 1922 Apprentice Engr. on Varattar Bridge Construction in Travancore State, South India, setting out, supervision, etc. *TT 0.3: SP 0.3.*—Oct. 1922 to Aug. 1923 Probationery Engr. with Jackson & Barker, Archts. and Civ. Engrs., Madras, India, on design, quantity surveying, stress calculations, valuation report, etc., and field inspection of building construction. *TT 0.5: SP 0.5.*—Sept. to Oct. 1923 Cons. Engr. to Methodist Episcopal Mission and Wesleyan Mission of Madras, on designs and supervision of alterations and additions to schools, bungalows, printing press and hospital. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—Nov. 1923 to July 1926 Constructional Asst. Engr., Gen. Constr. Co., Ltd., Archts. and Bldrs., Madras, in charge of construction of palaces, public buildings, etc., design of sanitary installation, surveys, etc. *TT 2.7: P 2.7: RC 2.7: D 0.5.*—Aug. 1926 to Aug. 1928 Res. Engr., Imperial Bank of India, in charge of construction of office and quarters for agent in Tuticorin, execution and detail design, alterations and additions to bank's property in other towns, etc. *TT 2: P 2: RC 2: D 0.5.*—Sept. to Nov. 1928 Constr. Engr., Gannon, Dunkerley & Co., Ltd,, Bombay, India, in charge of branch in Madras Presidency, on design and erection of structural work. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—Dec. 1928 to Jan. 1930 Asst. Engr., Public Works Dept., Govt. of Ceylon, in charge of office of and Personal Asst. to Provincial Engr., Sabragamuwa, (annual expenditure about Rs. 1 750 000); also Engr. in charge of Ratnapura Water-Supply Scheme (Rs. 414 000). *TT 1: P 1: RC 1: D 0.7.*—Jan. to May 1930 Special Engr., Urban Dist. Council of Ratnapura, investigation for town improvement, flood housing and drainage schemes, etc., involving contouring, surveying, designs, etc. *TT 0.4: P 0.4: RC 0.4.*—May 1930 to Sept. 1931 Engr., C. A. Hutson & Co., Ltd., Colombo, in charge of water-works, large building construction, etc. (approx. Rs. 1 000 000). *TT 1.3: P 1.3: RC 1.3: D 0.7.*—Sept. 1931 to date Asst. Engr., Gen. Constr. Co., Ltd.,

Engrs. and Contrs., Madras, in charge of Victory Memorial Works, etc. *TT 0.5: P 0.5: RC 0.5.*—*TT 10.2: SP 0.8: P 9.4: RC 8.4: D 3.3.* Refers to H. G. Howard, R. D. N. Simham. (Applies in accordance with Sec. 1, Art. 1, of the By-Laws.)

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(1) **FOX, FREDERICK JAY,** 1881 Walton Ave., Bronx, N. Y. (Age 37. Born Boston, Mass.) Registered Prof. Engr., New York State.—Student, Cooper Union (1911 to 1912, nights) and Columbia Univ. (1912 to 1913).—1914 to 1917 with Geo. F. Pelham, Archt., New York City, as Draftsman, Steel Designer, Specification Writer, Dept. Man and (after 1915) Chf. Draftsman, checking drawings and interpreting plans, laying out works, etc.; designed cantilever foundation for a 6-story building for West Side Constr. Co. *TT 2.2: SP 0.7: P 1.5: RC 1.5: D 1.5.*—1917 to 1919 with U. S. Constr. Q. M., Washington, D. C., at Homestead Works, Carnegie Steel Co., Pittsburgh, Pa., on Brest (France) signal towers and design and checking of Brooklyn Army Base, also Virginia Nitrate Plant, being Civilian Eng. Inspector.—*TT 2: P 2: RC 2: D 0.5.*—1919 to 1923 Chf. Draftsman with Margon & Glasser, New York City, steel design, outside supervision, contract letting and general construction, on Muriel Arms apartment house, Kingsbridge Rd. and Grand Concourse, etc.; designed, and checked details, for steel and foundations for about twenty-five jobs. *TT 4: P 4: RC 4: D 4.*—1923 to 1924 Estimator and Bldg. Appraiser on several jobs for World Acceptance Corporation, New York City. *TT 1: P 1: RC 1.*—1925 to 1927 with Sam'l Weiskopf, New York City, as Arch. and Eng. Designer on Navarre and Windham Hotels and two apartment houses, checked details and acted as Outside Supervisor on Bronx Hospital, also on underpinning design for a section of subway for Rosoff Bros., Brooklyn, N. Y. *TT 1.5: P 1.5: RC 1.5: D 0.5.*—1927 to 1929 Chf. Engr., Kaufman & Snyder Co., on male barracks, Welfare Island, two apartment houses and one foundation job in Yonkers, N. Y. *TT 2.5: P 2.5: RC 2.5: D 0.2.*—1929 to date Secy., Arthur I. Kraft, Inc., Engr. Contrs., New York City, completed 57th to 59th St. approach to Queensboro Bridge, for Dept. of Plant and Structures, test piles, 23d to 26th St. for elevated express highway, for Borough Pres. of Manhattan, and at present on completion of South Oceanic wing for American Museum of Natural History. *TT 3: P 3: RC 2.5: D 0.5.*—*TT 16.2: SP 0.7: P 15.5: RC 15: D 7.2.* Refers to E. A. Byrne, A. Dick, J. Feld, J. J. Murphy, B. Schwerin.

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(11) **GIBSON, JOHN BRUCE,** 1933 Casa Grande St., Pasadena, Cal. (Age 25. Born Knox, Pa.) 1932 B. S. in C. E., Univ. of S. Cal. *TT 4: P 4.* Refers to R. M. Fox, D. M. Wilson.

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(11) **GOODRIDGE, RICHARD SAMUEL,** 2636½ South Bronson Ave., Los Angeles, Cal. (Age 34. Born Chicago, Ill.) Student Coll. of Eng., Univ. of Minn. (Sept. 1919 to June 1921) and Univ. of Wis. (Sept. 1921 to Feb. 1923.) *TT 1.5: P 1.5.*—April to Sept. 1919 Draftsman, Meriden Iron Min. Co., Coleraine, Minn., chaining, rodding, drafting, estimating, etc. *TT 0.2: SP 0.2.*—Feb. 1923 to April 1924 Draftsman and Res. Engr., with Wisconsin State Highway Comm. as Draftsman on plans, profiles and estimates of highways on State and Federal Aid Projects and Res. Engr. on construction and inspection of gravel and concrete roadway, final cross-sections and estimates. *TT 1: SP 0.1: P 0.9: RC 0.7: D 0.2.*—May 1924 to June 1927 Draftsman, Los Angeles County Road Dept., drafting, plans and profiles, design of drainage structures, estimates of highway and street pavements. *TT 2.3: SP 0.8: P 1.5: RC 1: D 1.5.*—June 1927 to date Hydrographer, Los Angeles County Flood Control Dist., on hydraulic and hydrographic studies of superficial run-off, on investigation of rainfall-run-off relations, routing of theoretical flood flows and reports. *TT 4.8: P 4.8: RC 4.1: D 4.8.*—*TT 9.8: SP 1.1: P 8.7: RC 5.8: D 6.5.* Refers to E. A. Burt, J. H. Dockweiler, E. C. Eaton, F. F. Friend, F. Gillelen, F. Thomas.

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(4) **GREER, ROBERT COLLINS, Jr.,** 34 Shelburne Road, Springfield, Pa. (Age 37. Born Merchantville, N. J.) 1922 B. Sc. in C. E., Ohio Northern Univ. *TT 4: P 4.*—1912 to 1917 student (evening classes), Drexel Inst.—Sept. 1912 to April 1917 Chainman to Chf. of Party with Damon & Foster, Sharon Hill, Pa., on surveys, laying out subdivisions, surveying and construction of streets, roads, curb and gutter, sewerage system, disposal plants, bridges, retaining walls, bulk head lines and river soundings. *TT 2.7: SP 2: P 0.7.*—April 1917 to May 1919 with 103d Engrs., 28th Div., U. S. Army, from Private to 2d Lieut., in charge of field work of topographic surveys, Camp Mead, Md. and Camp Hancock, Ga., constructed military bridge and roads in France, also balloon observation. *TT 2: P 2: RC 2.*—Summers 1919–1921 and June 1922 to Oct. 1924 Asst. Engr. with Damon & Foster, surveying, constructing and designing of subdivision, roads, bridges, disposal plants and sewerage systems. *TT 2.6: P 2.6: RC 2: D 1.2.*—Oct. 1924 to

March 1930 Engr. and Contr., designing and constructing sewers and water systems, roads, pavements, retaining walls and grading. *TT 5.5: P 5.5: RC 5.5.*—March to Nov. 1930 Engr. in charge of field parties for Damon & Foster, on location, preliminaries, construction and final survey of Susquehanna pipe line, Philadelphia to Ohio and Philadelphia to Syracuse, N. Y. *TT 0.7: P 0.7: RC 0.7.*—Nov. 1930 to date Civ. Engr., Susquehanna Pipe Line and Sun Pipe Line Companies, in charge of all civil engineering work. *TT 1.5: P 1.5: RC 1.5.*—*TT 19: SP 2: P 17: RC 11.7: D 1.2.* Refers to J. H. M. Andrews, A. F. Damon, Jr., G. H. Elbin, C. Elcock, N. Foster, F. A. Snyder.

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(1) **HENRY, MAXWELL,** 222 East Fifteenth St., New York City. (Age 34. Born New York City.) 1920 B. S., and 1924 C. E., Coll. of City of N. Y. *TT 4: P 4.*—Prior to 1920 (vacations and spare time while student) on various work, including some design and building of electrical and mechanical equipment, assisting on layout of footing for School of Technology, Coll. of City of New York, etc.—1920 to date with Coll. of City of New York, until 1924 taught rehabilitation work; wrote entire course, job sheets, etc., for Elec. Wiring Dept.; laid out, purchased, installed and maintained electrical appliances for all units of college in Brooklyn and Manhattan; appointed Expert Examiner for City of New York, prepared examinations, appointed assistant examiners, rated questions, etc.; since 1924 Instructor in Eng., Elec., Civ. and Mech. Depts., School of Technology; 1924-1925 laid out and installed electrical engineering laboratories; 1925 laid out Hydraulic Laboratory and taught advanced surveying; since 1929 with Prof. H. Baum, investigated plant, made plans, wrote specifications, supervised work (first unit completed, cost $50 000, second unit under construction, $150 000, third unit now being laid out); assisted on Brooklyn City Coll. buildings; made preliminary surveys for Hunter Coll.; has served on college engineering committees. *TT 11: P 11: RC 11: D 11.*—*TT 15: P 15: RC 11: D 11.* Refers to R. E. Goodwin, F. O. X. McLoughlin, G. Paaswell, J. S. Peck, J. C. Rathbun, D. B. Steinman.

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(16) **HIPP, CHARLES LEO,** 3616 Olive St., Kansas City, Mo. (Age 22. Born Kansas City, Mo.) 1931 B. S. in C. E., Kans. Univ.. .*TT 4: P 4.*—Aug. 1931 to date Rodman and Draftsman, U. S. Engr. Office, Kansas City, Mo., on computations, mapping and valuation. *TT 0.4: SP 0.4.*—*TT 4.4: SP 0.4: P 4.* Refers to W. C. McNown, H. A. Rice, F. A. Russell, A. R. Young.

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(15) **HIPP, THOMAS MARTIN,** Box 327, Whittenburg, Tex. (Age 27. Born Kansas City, Mo.) 1926 B. S. in Civ. Eng., Univ. of Kans. *TT 4: P 4.*—June 1926 to Feb. 1932 with Phillips Petroleum Co., about 10 months as Instrumentman and Draftsman, 8 months (March to Nov. 1928) Res. Constr. Engr., Tex-Roy Gasoline Plant, on layout and installation of plant equipment, pipe-line surveys and construction, etc., and over 4 years Constr. Engr., Oklahoma and Texas Dists., installing air and gasoline plants, tank farms, pipe lines, etc., surveys, tank farm designs, laying out and installing equipment at refinery and gasoline plants, including layout and installation of stills, loading racks, pipe lines, camps, tanks, grades, pipe work and fittings, design of concrete and pipe work, field plans, etc., for Alamo refinery, also addition to gasoline plants, pipe line surveys, etc. *TT 5.3: SP 0.4: P 4.9: RC 4.9: D 2.2.*—*TT 9.3: SP 0.4: P 8.9: RC 4.9: D 2.2.* Refers to L. Arnold, H. W. Crawford, H. A. Rice, A. H. Riney, F. A. Russell.

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(7) **HUBKA, VENCEL ANTHONY,** 512 South Union St., Traverse City, Mich. (Age 24. Born Traverse City, Mich.) 1931 B. S. in Civ. Eng., Mich. Coll. of Min. and Tech. *TT 4: P 4.* Refers to H. B. Pettit, W. C. Polkinghorne.

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(13) **JOHNSON, IRVING LAURENCE,** 2643 Portola Way, Sacramento, Cal. (Age 25. Born Sacramento, Cal.) Student in Civ. Eng., Sacramento Junior Coll. (Sept. 1924 to June 1926) and Stanford Univ. (Oct. 1926 to March 1928) and Summer Surveying course, Univ. of California (1925). *TT 0.5: SP 0.5.*—June 1928 to date Res. Engr. and Designer, Dept. of Eng., Sacramento County, Cal., on work, including construction of bascule bridge over Sacramento River at Freeport, Cal. ($240 000), design on substructure for and construction of drawbridge over American River at Sacramento ($240 000), design and construction of, and Shop Inspector for structural steel and machinery for, drawbridge over Snodgrass Slough ($50 000), etc., and at present on design for bridge on Fair Oaks Boulevard over American River. *TT 3.7: P 3.7: RC 3.5: D 0.7*—*TT 4.2: P 4.2: RC 3.5: D 0.7.* Refers to D. Butler, B. C. Gerwick, J. W. Gross, A. G. Proctor, D. R. Warren.

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(15) KILLMER, ROBERT EDWARD, Box 207, Refugio, Tex. (Age 43. Born Sandia, Tex.) 1913 C. E., Univ. of Tex. *TT 4: P 4.*—June 1913 to April 1914 Rodman, Isthmian Canal Comm., Dredging Div., Panama Canal, Balboa, Canal Zone. *TT 0.4: SP 0.4.*—April 1914 to Sept. 1919 not on engineering work.—Sept. 1919 to March 1923 and Nov. 1925 to Nov. 1926 with Bexar County, San Antonio, Tex., until Dec. 1919 as Draftsman, then (1 year) Office Engr., in office of County Highway Engr., responsible for detail work on road building plans, Dec. 1920 to July 1922 Asst. County Highway Engr., assisting in highway maintenance and construction; July 1922 to March 1923 County Highway Engr., in charge of highway construction and maintenance, being directly responsible to County Commissioners' Court, and after Nov. 1925 Res. Bridge Engr., under County Highway Engr., in charge of construction of highway bridges. *TT 4.4: SP 0.1: P 4.3: RC 0.7: D 0.1.*—March 1923 to July 1925 County Engr., Frio County, Tex., in direct charge of construction of 48 miles of paved roads. *TT 2.3: P 2.3: RC 2.3: D 1.1.*—Nov. 1926 to date Res. Engr., Texas State Highway Dept., until Nov. 1927 in charge of construction, including paving of 8-mile project in Bexar County, and since Nov. 1927 in charge of location, design and construction of 67 miles of high type paved state highway in Refugio County. *TT 5.4: P 5.4: RC 4.4: D 2.5.*—*TT 16.5: SP 0.5: P 16: RC 7.4: D 3.7.* Refers to T. W. Bailey, G. G. Edwards, G. Gilchrist, M. B. Hodges, T. E. Huffman, H. C. Porter, T. U. Taylor, G. G. Wickline.

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(1) KO, CHOONG MYUNG, 158 Ellison St., Paterson, N. J. (Age 33. Born Song Do, Korea.) 1926 B. S., Mass. Inst. Tech. *TT 4: P 4.*—July 1926 to Feb. 1927 Draftsman for Edmund R. Halsey, Civ. Engr. and Surveyor, Newark, N. J., calculating and drafting surveys, estimating cut and fill for road work and street planning. *TT 0.5: SP 0.2: P 0.3: RC 0.3.*—Feb. 1927 to Dec. 1930 Asst. Engr., Passaic Consolidated Water Co., on river flow, surveying and mapping, designing water pipe lines, small bridges, filters, intake screens and buildings, work on water-distribution system, water-tanks and pumps, also testing water mains. *TT 3.4: SP 0.5: P 2.9: RC 1.4: D 1.5.*—Aug. 1931 to date Engr. in field for Frank A. Barbour and Weston E. Fuller, Cons. Engrs. for Passaic Valley Water Comm., on construction of pumping station, installing hydro-electric turbines, pumps and pipes. *TT 0.5: P 0.5: RC 0.3: D 0.3.*—*TT 8.4: SP 0.7: P 7.7: RC 2: D 2.* Refers to H. K. Barrows, A. T. Cook, J. H. Cook, W. E. Fuller, G. T. Seabury, R. G. Tyler.

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(4) LA ROSEE, PAUL HENRI, Box 16, Quarryville, Pa. (Age 28. Born Philadelphia, Pa.) Dec. 1920 to Dec. 1922 Sergeant, U. S. Marine Corps, being student and instructor in Civ. Eng. and Navigation Schools, U. S. Marine Corps Inst., Washington, D. C.; received certificate of proficiency in Mathematics and Mechanics and completed Surveying and Mapping course.—April 1923 to Aug. 1925 Chf. of Party, Pennsylvania Water & Power Co., on steel-tower transmission line (58 miles) and highway location and construction, power-plant construction, town planning and layout, etc. *TT 1.6: SP 0.8: P 0.8: RC 0.8.*—Aug. 1925 to Sept. 1926 Draftsman, Venezuela Gulf Oil Co., Barcelona, Venezuela, topographical drafting, surveys, planetable work, triangulation, etc. *TT 1.1: P 1.1: RC 0.3.*—Oct. 1926 to July 1928 Asst. Field Engr., Day & Zimmerman Eng. & Constr. Co., on surveys, Conowingo-Philadelphia 220-kv. transmission line, 53 miles of double line, last 5 months on photographic survey of adjacent telephone and power lines. *TT 1.7: P 1.7: RC 1.7.*—July 1928 to Aug. 1929 Asst. Engr., United Fruit Co., Guatemala, C. A., on exploration surveys, railroad engineering, designing, irrigation, drainage, town planning and layout, etc. *TT 1.1: P 1.1: RC 1.1: D 0.2.*—Sept. 1929 to March 1930 Inspector of Surveys, Corps of Engrs., U. S. Army, U. S. Quarterboat 14, Melville, La., in charge of engineer office on quarterboat, topographic surveys for location of levees, levee location and right-of-way surveys, design of survey equipment, Atchafalaya Floodway Basin. *TT 0.5: P 0.5: RC 0.5: D 0.1.*—March 1930 to date Asst. Field Engr., Pennsylvania Water & Power Co., on 150 miles of 220-kv. transmission line surveys (steel towers), layout and inspection of hydraulic testing laboratory at Holtwood, Pa., inspection and removal of timber in flowage basin, Safe Harbor Reservoir, design of survey equipment. *TT 2: P 2: RC 2: D 0.1.*—*TT 8: SP 0.8: P 7.2: RC 6.4: D 0.4.* Refers to C. M. Africa, F. A. Allner, F. F. Henshaw, J. T. Kiernan, H. E. Whitney.

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(9) McKINNEY, GERALD FRANCIS, 1022 Kinney's Lane, Portsmouth, Ohio. (Age 40, Born Dayton, Ohio.) 1910 B. S. in C. E., Univ. of Dayton. *TT 4: P 4.*—1910 to 1911 Post-graduate student, Univ. of Dayton.—Sept. 1911 to March 1914 Chainman, Rodman

and Instrumentman, Norfolk & Western Ry. Co. *TT 1.2: SP 1.2.*—March 1914 to April 1915 Asst. Res. Engr., Ohio State Highway Dept., in charge of field parties and developing plans and estimates. *TT 1: P 1: RC 0.5: D 0.5.*—April 1915 to Sept. 1916 Res. Engr., Ohio Valley Traction Co., Portsmouth, Ohio, in charge of construction, Res. No. 3 *TT 1.5: P 1.5: RC 1.5: D 1.5.*—Sept. 1916 to Oct. 1917 and June 1920 to Sept. 1923 with Portsmouth Works, Wheeling Steel Corporation, as Field Engr. in charge of layouts and supervision of construction, including construction of rod and wire mill, and (after July 1922) Estimator for new construction and industrial improvements. *TT 4.2: P 4.2: RC 4.2: D 1.2.*—Oct. 1917 to August. 1919 Private, etc., and finally 1st Lieut., 308th and 34th Engrs., U. S. Army. *TT 1.7: P 1.7: RC 1.7.*—Aug. 1919 to Feb. 1920 with Office of County Surveyor, Scioto County, developing plans and estimates for roads and bridges. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—Feb. to June 1920 with Vulcan Last Co., Portsmouth, supervising, wrecking and rebuilding industrial buildings. *TT 0.3: P 0.3: RC 0.3.*—Sept. 1923 to Feb. 1927 Deputy County Surveyor, Scioto County, and Asst. Highway Engr., State of Ohio, developing plans and estimating State highways and bridges in Scioto County. *TT 3.5: P 3.5: RC 3.5: D 3.5.*—Feb. 1927 to date Sales Engr., Peebles Paving Brick Co., Portsmouth. *TT 5.1: P 5.1: RC 5.1.*—*TT 23.1: SP 1.2: P 21.9: RC 17.4: D 7.2.* Refers to H. D. Bruning, J. W. Graham, W. W. C. Perkins, T. A. Polansky, R. S. Proctor, G. F. Schlesinger, P. S. Wickerham, C Wuest, Jr.

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(1) MICHELS, WILLIAM, 4294 Oneida Ave., New York City. (Age 39. Born New York City.) Registered Prof. Engr., N. Y. State.—1915 B. S. in C. E., Cooper Union.—Oct. 1912 to Oct. 1917 Jun. Engr., Public Service Comm., First Dist., New York State, on subway construction, being Instrumentman, Chf. of Party, Draftsman, Inspector and Estimator. *TT 3: SP 2: P 1: RC 1.*—Oct. 1917 to Jan. 1918 Asst. Engr., DuPont de Nemours Co., estimating, equipment installation and layout design. *TT 0.1: SP 0.1.*—Jan. 1918 to June 1920 Inspector of construction, U. S. Navy Dept., Bureau of Yards and Docks, on surveys, drafting, design, estimating and inspection. *TT 1.7: SP 0.7: P 1: RC 1.*—June to Dec. 1920 Asst. Engr., J. G. White Eng. Corporation, being Designer and Draftsman. *TT 0.5: P 0.5: D 0.5.*—Dec. 1920 to July 1921 Utilities Engr., U. S. Army, Chemical Warfare Service, in charge of artillery proving grounds, camp utilities and appraisal surveys. *TT 0.7: P 0.7: RC 0.7.*—July 1921 to Feb. 1923 Instructor, Coll. of City of New York, taught civil engineering subjects to classified war veterans. *TT 1.5: RC 1.5: D 1.5.*—Feb. 1923 to Oct. 1924 Supt. and Engr. with Richard Carvel, Gen. Contr., New York City, building design, estimates and executive; designed and built a garage for Glidden Buick Co., The Stewart-Warner Bldg. and Automobile Club of America Garage. *TT 1.8: P 1.8: RC 1.8: D 1.*—Oct. 1924 to Oct. 1927 Engr., Kenlon-Michels Co., Inc., Contrs. and Engrs., New York City, being Gen. Supt. and Executive; did all engineering and estimating, built gasoline service stations, alterations to theatres, designed and built plant of Standard Oil Co. of N. Y., etc. *TT 3: P 3: RC 1: D 2.*—Oct. 1927 to July 1929 Engr. and Contr., Edgewater Constr. Co., Inc., Contrs. and Engrs., being Gen. Supt. and Executive; had charge of all field engineering on work ($1 000 000), including a terminal building, addition to an armory, alterations to theatre buildings, etc., a church, a garage, etc. *TT 1.7: P 1.7: RC 0.7: D 1.*—July 1929 to date Engr. and Contr., New York City, on engineering work and contracting, including altering loading and storage yard into garage and a yard office building for Standard Oil Co. of N. Y., alteration on Marine Hospital No. 10, New York City, etc., also Cons. Engr. for Hercules Constr. Co., Valentine Constr. Co., Giraud & Welsch, Civ. Engrs., etc. *TT 2.7: P 2.7: RC 0.7: D 2.*—*TT 16.7: SP 2.8: P 13.9: RC 7.4: D 6.5.* Refers to E. Brittan, A. E. Clark, P. M. Corry, C. F. Giraud, J. F. Krakauer, F. O. X. McLoughlin, C. Pierleoni.

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(15) MIMS, STALEY WOOD, East Third St., Crockett, Tex. (Age 28. Born McGregor, Tex.) 1925 B. S. in C. E., A. & M. Coll. of Tex. *TT 4: P 4.*—1925 to 1927 with St. Louis South Western Ry., successively as Chainman, Rodman, Levelman, Transitman and Acting Asst. Engr. (9 months) and Sr. Transitman (6 months). *TT 1.2: SP 0.7: P 0.5.*—1927 to 1929 Instructor (1 year) and Asst. Prof., Southern Methodist Univ. Eng. School, in charge of Drawing Dept. and assisting in Mathematical Dept. *TT 2: P 2.*—1929 to date Instrumentman, acting as Field Asst. to Res. Engr., Texas State Highway Dept. *TT 2.5: P 2.5: RC 1.5.*—*TT 9.7: SP 0.7: P 9: RC 1.5.* Refers to E. B. Darby, T. G. MacCarthy, J. T. L. McNew, E. J. Nichols, J. J. Richey.

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(1) MORALES, JOSE DIONISIO, Post St., Mayaguez, Porto Rico. (Age 39. Born Naranjito, Porto Rico.) 1917 C. E., and 1928 M. S., Rens. Pol. Inst.. *TT 4: P 4.*—July to Sept. 1917 Draftsman. McClintic Marshall Co., Pittsburgh, Pa. *TT 0.1: SP 0.1.*—Oct. 1917–Jan. 1918 in private practice of surveying in Porto Rico. *TT 0.3: P 0.3: RC 0.3.*—Feb. to June 1918 Draftsman, and July to Dec. 1921 Assst. Engr., Bridge Dept., Dept. of Interior, San Juan, Porto Rico. *TT 0.7: SP 0.2: P 0.5: D 0.5.*—July to Dec. 1918 with War Dept., Third Officers' Training Camp, Washington, D. C.—Jan. 1919 to June 1921 in private practice of surveying (⅓ of time) and teaching elementary mathematics in secondary schools in Porto Rico (⅔ of time) being Principal, Guayama High School (6 months). *TT 1.7: SP 0.9: P 0.8: RC 0.8.*—July 1927 to June 1928 graduate student, Rensselaer Polytechnic Inst.—Jan. to June 1922 Asst. Prof., and July 1922–June 1927 and June 1928 to date Prof., of Mathematics, Coll. of Agriculture and Mechanic Arts, Mayaguez, Porto Rico; since Jan. 1928 Head of Eng. Faculty. *TT 9.1: P 9.1.*—*TT 15.9: SP 1.2: P 14.7: RC 1.1: D 0.5.* Refers to R. R. Casellas, L. W. Clark, M. Font, R. Ramirez, P. C. Ricketts, E. Totti y Torres.

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(1) MORRISON, JOHN H., 626 Carlton Ave., Brooklyn, N. Y. (Age 25. Born Brooklyn, N. Y.) 1932 B. S. C, E., Univ. of Mich. *TT 4: P 4.*—Sept. 1925 to June 1927 Estimator, until Feb. 1927 with Brooklyn Edison Co., on computations of costs of removal, then with Asbestile Co. (Brooklyn), on cost of ornamental (tile, slate, asbestos) roofing. *TT 0.9: SP 0.9.*—June to Sept. 1927 Sales Engr. with Jos. T. Ryerson, (steel service), Jersey City, N. J., estimation from plans and selling. *TT 0.1: SP 0.1.*—Summer 1928 Transitman with D. Baswell, Ridgefield Park, N. J.—*TT 5: SP 1: P 4.* Refers to W. W. Rousseau, W. C. Sadler, J. S. Worley.

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(1) MOTION, ROBERT, 1 Woodside Road, Madison, N. J. (Age 22. Born New York City.) 1931 B. S., Lehigh Univ. *TT 4: P 4.* Refers to S. A. Becker, R. J. Fogg, H. G. Payrow, C. H. Sutherland, E. H. Uhler.

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(14) NEWMAN, JAMES ROY, 79 North Bellevue St., Memphis, Tenn. (Age 35. Born Helenwood, Tenn.) Sept. 1914 to March 1917 student, Univ. of Tennessee. *TT 1: P 1.*—July 1920 to Dec. 1921, May 1922 to May 1927 and June 1929 to March 1930 with R. H. Randall & Co., Toledo, Ohio, until May 1923 as Draftsman, Computer, Field Asst. and Office Engr., on map drafting, surveys, property checking, computing and adjusting triangulation, traverse and levels, compiling property atlas, etc., part of time being Chainman, Transitman and Recorder; June 1923 to May 1927 Res. Engr. on topographic, geodetic and cadastral surveys in Durham and Greensboro, N. C., and West Palm Beach, Fla., in local charge of field and office on triangulation, traverse and levels, topographic maps, etc., being responsible for design of control and for map layouts, and co-responsible for specifications and reports; after June 1929 Office Engr. on topographic survey of part of Willacy County, Tex., in charge of office work, including design of control, computations, drafting, property chceking, etc., and co-responsible for revisions of specifications. *TT 6.2: SP 1.8: P 4.4: RC 4: D 1.2.*—March to Oct. 1928 Draftsman, City of Greensboro, N. C., general map and plat and plan drafting for Tax, Water and Street Improvement Depts., design of street and park layouts in undeveloped areas for Planning Comm. *TT 0.4: SP 0.3: P 0.1: D 0.1.*—May 1930 to date with U. S. Engr. Office, until Nov. 1931 as Senior Draftsman at Norfolk, Va., drafting, computing, making economic studies for flood control, historical reports on early navigation, special study and report on a comparison of reservoir volume curves, design of a control system for aeroplane mapping and supervision of computations for map control, etc., and since Dec. 1931 Senior Draftsman on river improvement. *TT 1.3: SP 0.6: P 0.7: D 0.2.*—*TT 8.9: SP 2: P 6.9: RC 4: D 1.5.* Refers to G. H. Matthes, F. T. Miller, R. H. Randall. T. K. Roberts, N. H. Sayford, C. W. Smedberg, G. D. Whitmore.

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(9) NOE, CHARLES LAURIN, 1860 Winfield Ave., Indianapolis, Ind. (Age 26. Born Indianapolis, Ind.) 1932 B. S. in C. E., Purdue Univ. *TT 4: P 4.*—Aug. to Nov. 1928 Surveyor, Chainman and Rodman with H. O. Garman, Cons. Engr., Indianapolis, Ind., on subdivision work. *TT 0.1: SP 0.1.*—Nov. 1928 to March 1929 Instrumentman, Illinois Pipe Line Co., on surveys. *TT 0.2: SP 0.2.*—*TT 4.3: SP 0.3: P 4.*—Refers to H. O. Garman, W. K. Hatt, G. E. Lommel.

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(9) NYQUIST, ROY ALFRED, 201 Belmont Ave., Toledo, Ohio. (Age 27. Born Minneapolis, Minn.) 1927 B. S. in Arch. Eng., Univ. of Minn. *TT 4: P 4.*—April to Aug. 1927 Arch. Draftsman with O'Meara & Hills, Archts., St. Paul, Minn., and Aug. to Sept. 1927 Structural Draftsman with Croft & Boener, Archts., Minneapolis, Minn., on plans and details for high-school buildings, etc. *TT 0.3: SP 0.3.*—Nov. 1928 to Aug. 1929 Arch. Engr., The Edward Ford Plate Glass Co., Rossford, Ohio, being Asst. in charge of structural and architectural designs and plans for concrete and steel factory buildings, including grinder and polisher and motor generator buildings, boiler house, power-house extension, etc. *TT 0.8: P 0.8: RC 0.8.*—Sept. 1927 to date (except Nov. 1928 to Aug. 1929 on leave) Arch. Engr., A. Bentley & Sons Co., Engrs. and Contrs., Toledo, Ohio, on structural designs, plans and estimates for concrete, steel and wood factory and office buildings, etc., including commercial buildings, warehouses, garage, boiler house and coal-receiving pocket, boathouse, retaining walls for boat channel, ice breaker for an estate, a die casting building, for alteration of a glass plant into storage warehouse, and for alteration of wing of main building of Toledo Museum of Art, consolidating three galleries into one cloister gallery, providing new trusses and columns to carry roof, and reinforcing main floor structure, etc. (detailed statement forwarded with application). *TT 3.8: SP 0.1: P 3.7: RC 2: D 1.*—*TT 8.9: SP 0.4: P 8.5: RC 2.8: D 1.* Refers to R. P. Barber, R. B. Daudt, A. Gardner, G. A. Maney, J. I. Parcel, C. B. Patterson, E. I. Roberts.

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(6) OLAFSEN, REIDAR, Glen Ferris, W. Va. (Age 32. Born Mosjoen, Norway.) 1926 C. E., Tech. Univ. of Norway. *TT 4: P 4.*—Aug. 1921 to Feb. 1924 Draftsman, Govt. Rys., Narvik Dist., Norway, being Traffic Inspector and Elec. Inspector, drafting, surveying transmission lines, estimating, etc. *TT 1.2: SP 1.2.*—April to Sept. 1927 Bridge Detailer, Bridge Dept., New Jersey State Highway Comm., on structural steel and reinforced concrete bridges. *TT 0.4: P 0.4.*—Sept. 1927 to March 1929 Bridge Designer, 6 months with Michigan State Highway Dept., designing, and 1 year with Connecticut State Highway Dept., designing and supervising drafting of, structural steel and reinforced concrete bridges, including a 250-ft. arch bridge. *TT 1.5: P 1.5: D 1.5.*—March 1929 to date Structural Designer, New Kanawha Power Co. (subsidiary of Union Carbide & Carbon Corporation), Glen Ferris, W. Va., desiging and supervising drafting of structural steel and reinforced concrete structures. *TT 3.1: P 3.1: RC 1: D 2.*—*TT 10.2: SP 1.2: P 9: RC 1: D 3.5.* Refes to C. S. Bissell, L. H. Davis, M. Goodkind, J. W. Hall, O. M. Jones, C. A. Melick.

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(12) PECORE, CHESTER WENTWORTH, P. O. Box 156, Burns, Ore. (Age 35. Born Port Angeles, Wash.) 1929 L. L. B., National Univ., Washington, D. C.—May to June 1917 Levelman and Transitman, Office of City Engr., Blythe, Cal., on surveys and irrigation work. *TT 0.1: SP 0.1.*—March 1918 to Jan. 1919 Private, Corporal and Sergeant, Eng. Corps, U. S. Army, Miners and Sappers, on barbed-wire laying, trench and bridge construction, etc. *TT 0.5: SP 0.5.*—June 1915 to date (except as above noted) with U. S. Gen. Land Office, in Arkansas, Arizona, Washington, California, and Washington, D. C., until Nov. 1924 on public land surveys as Axeman, Rodman, Chainman, Flagman, Levelman, Transitman, Associate Transitman, Prin. Asst. and (about 5 years) Chf. of Field Party, executed surveys, prepared field notes, reports, etc.; Dec. 1924 to May 1925 Examiner of surveys, passing on field notes and plats for acceptance of surveys by Comm.; June to Sept. 1925 and March to June 1926 Chf. of Field Party on surveys of Islands, etc.; July 1926 to March 1931 in office of Commr., Washington, D. C., authorizing preliminary field investigations, reviewing special instructions, inspecting and making detailed examination of final returns on surveys and resurveys, recommending acceptance or rejection, conducting research on cadastral engineering problems, preparing memoranda and reports, etc.; Oct. 1925 to Feb. 1926 and since April 1931 Cadastral Engr., supervising field work of engineering parties of subsidiary of Elec. Bond & Share Co., establishing flow-line of hydro-electric reservoir, Spokane (Wash.) Indian Resevation, and (since April 1931) in charge of cadastral and topographical surveys for suit, United States *vs.* Oregon, involving title to beds of Malheur, Mud and Harney Lawes, Harney County, Ore. *TT 14.6: SP 1.1: P 13.5: RC 7: D 4.8.*—*TT 15.2: SP 1.7: P 13.5: RC 7: D 4.8.* Refers to F. M. Johnson, A D. Kidder, W. H. Richards, Jr., J. C. Thoma, C. G. Tudor.

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(1) PORCHER FRANCIS DAVIS, 227 West Seventieth St., New York City (Age 34. Born St. Louis, Mo.) Prof. Engr. and Land Surveyor, New York State.—1917 B. S. in C. E., Va. Mil. Inst. *TT 4: P 4.*—Summer 1917 surveying, Massachusetts Inst. of Technology.

—Oct. 1918 to Dec. 1921 various activities, including service in Naval aviation, student (1 year) in Business Administration at Yale Univ., Asst. Supt. in Boston Navy Yard and Seaman on Munson Line.—March to Oct. 1918 Draftsman, Hood Rubber Co., Watertown, Mass., on building construction, Dec. 1921 to April 1922, Chainman, Atchison, Topeka & Santa Fe Ry., Chicago, Ill., and Sept. to Dec. 1922 Rodman, Massachusetts Dept. of Highways. *TT 0.6: SP 0.6.*—Dec. 1922 to July 1923 Transitman, United Fruit Co., Guatemala development. *TT 0.6: P 0.6: RC 0.6.*—Sept. 1923 to April 1924 Transitman with Stone & Webster, on power-house construction, Long Beach, Cal. *TT 0.5: P 0.5: RC 0.4.*—May 1924 to March 1925 Deck Officer, U. S. Coast and Geodetic Survey, chart-making, shore surveys, Pacific Coast. *TT 0.9: P 0.9: RC 0.1.*—April 1925 to April 1926 Surveyor and Draftsman, City of Los Angeles and Southern California Edison Co. *TT 0.9: P 0.9: RC 0.4.* —April 1926 to April 1927 Transitman, Computer, etc., on construction for Southern Pacific Ry. and W. B. Hoag, San Francisco. *TT 0.8: P 0.8: RC 0.8.*—June 1927 to May 1928 Inspector and Res. Engr., 6 months with Massachusetts Dept. of Highways, then with Warren Bros., on bituminous construction. *TT 0.6: SP 0.1: P 0.5: RC 0.4.*—May to Aug. 1928 Asst. Engr. with R. E. Horton, on topographical and soil surveys for water projects. *TT 0.3: P 0.3: RC 0.3.*—Sept. 1928 to July 1929 Structural Steel Designer and Transitman with Jackson & Moreland, A. A. Johnson, on Lackawanna R. R. electrification. *TT 0.8: P 0.8: D 0.5.*—July 1929 to March 1930 Chf. of Party, Inspector and Levelman with Stone & Webster, Long Island R. R. airport construction. *TT 0.6: P 0.6: RC 0.6.*—March to Oct. 1930 Eng. Inspector, Grade 4, Board of Water Supply, New York City, on tunnel construction. *TT 0.6: P 0.6: RC 0.6.*—Aug. to Oct. 1931 Draftsman, Dept. of Public Works, Borough of Manhattan, topographical and structural. *TT 0.2: P 0.2.*—Winter 1930-1931 Interviewer and Paymaster, Prosser Comm.'s Work Bureau; at present Chf. of Party on work for Public Works Dept., Borough of Manhattan.—*TT 11.4: SP 0.7: P 10.7: RC 4.2: D 0.5.* Refers to A. Dick, J. Friedland, W. B. Hunter, R. F. Luce, B. Schwerin, E. B. Whitman.

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(13) REIFF, JOHN ROBERT, 677 Fifty-third St., Oakland, Cal. (Age 24. Born Chantilly, France.) 1930 B. S., Ore. State Coll. *TT 4: P 4.*—June to Oct. 1930 Jun. Engr., acting as Draftsman, Standard Oil Co. of San Francisco. *TT 0.2: SP 0.2.*—*TT 4.2: SP 0.2: P 4.* Refers to J. R. Griffith, G. W. Holcomb, H. S. Rogers.

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(1) ROMERO, ANTONIO SILVANO, 3 Washington St., Santurce, Porto Rico. (Age 41. Born Barranquitas, Porto Rico.) 1912 S. B. Mass. Inst. Tech. *TT 4: P 4.*—1912 to 1914 with McClintic-Marshall Constr. Co., detailing, drafting, estimating, steel bridges, mill buildings, etc. *TT 0.7: SP 0.7.*—1914 to 1917 with Fajardo Sugar Co. in charge of Civ. Eng. Dept., surveys, extension or realignment of railroads, reinforced concrete, culverts and bridges, buildings, study of irrigation projects, etc. *TT 2.8: P 2.8: RC 2.8: D 0.5.*—1917 to 1919 with Dept. of Health of Porto Rico, in charge of San. Eng. Office; approved plans for residences, theatres, churches, storehouses, schools, water-works, sewerage systems, etc., controlled inspection, etc. *TT 2: P 2: RC 2.*—1919 to 1920 with Public Works Div., Dept. of Interior of Porto Rico, on design of plain and reinforced concrete culverts, etc. *TT 1: P 1: D 1.*—1920 to date with Public Service Comm. of Porto Rico, on valuations, rate studies, inspections, investigation, reports and in charge of public service utilities, including water-works, sewerage systems, electric light and power companies, railroads, gas companies, piers, bulkheads, water-power developments, etc. (approx. $43 650 000); made valuation of properties of American Railroad of Porto Rico, valuation and rate fixing, Porto Rico Telephone Co.'s rate case ($3 500 000). *TT 11.5: P 11.5: RC 11.5: D 0.5.*—*TT 22: SP 0.7: P 21.3: RC 16.3: D 2.* Refers to R. R. Casellas, J. V. Davila, M. Ferrer, M. Font, R. A. Gonzalez, R. Ramirez, E. Totti y Torres.

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(8) SANDBERG, CLIFFORD HELMER, 7752 Eastlake Terrace, Chicago, Ill. (Age 30. Born Minneapolis, Minn.) 1926 B. S. in C. E., 1929 M. S. in C. E., and 1931 C. E., Univ. of Minn. *TT 4: P 4.*—Oct. 1928 to June 1929 Graduate Student, Univ. of Minnesota, Teaching and Research Fellow.—March 1926 to Oct. 1928 Bridge Designer, and June 1929 to date Asst. Engr., Bridge Dept., Atchison, Topeka & Santa Fe R. R. *TT 5.3: P 5.3: RC 3.8: D 3.8*—*TT 9.3: P 9.3: RC 3.8: D 3.8.* Refers to A. S. Cutler, G. W. Harris, J. I. Parcel, H. Penn, R. A. Van Ness, C. E. Webb.

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(1) SOLAKIAN, ARSHAG GAZAR, 529 West One Hundred Twenty-third St., New York City. (Age 36. Born Gumush Hadji Keuy, Sivas, Turkey.) 1917 B. S. C. E., 1920 M. S. C. E., 1921 C. E., and 1922 B. S. M. E., Robert Coll., Constantinople, Turkey.

TT 4: P 4.—Sept. 1917 to Jan. 1930 Instructor, Eng. (Civ.) School, Robert Coll., from 1917 to 1929 had charge of Materials Testing Laboratory (Concrete and Metals); also 1920-1922 (summers and spare time) Draftsman-Designer, Docks and Dockyard of Stenia, Constantinople, Turkey, on design and supervision of a reinforced-concrete slipway, 1923-1927 occasional designs and construction supervision including a factory, steel roof frame, etc., and summer 1929 Designer, Rontaix et Maitrot, Paris, France, on reinforced concrete garages. *TT 12.5: P 12.5: RC 12.5: D 1.3.*—Jan. to April 1930 Graduate Student in Advanced Structures and Photoelasticity, London Univ.—April to Aug. 1930 Verifier of designs of reinforced concrete hangars for French Ministry of Air, Paris, also in charge of concrete-testing laboratory. *TT 0.3: P 0.3: RC 0.3.*—Aug. 1930 to date Lecturer in Mech. Eng., and (since Sept. 1931) Research Associate in Civ. Eng., Columbia Univ., New York City, in charge of Photoelastic Laboratory and Research. *TT 1.6: P 1.6: RC 1.6.*—*TT 18.3: P 18.3: RC 14.3: D 1.3.* Refers to A. H. Beyer, D. M. Burmister, W. J. Krefeld, C. H. Sutherland, H. C. Woods.

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(13) **SONNE, JULIUS ASA,** 1109 Capuchino Ave., Burlingame, Cal. (Age 23. Born Joseph, Ore.) 1932 B. S. in Civ. Eng., Univ. of Wash. *TT 4: P 4.*—Aug. 1930 to Jan. 1931 Asst., Concrete Control Laboratory, Stone & Webster Eng. Corporation, Rock Island Dam on Columbia River near Wenatchee, Wash., testing sand and gravel, designing concrete mixers, reports, etc. *TT 0.2: SP 0.2.*—*TT 4.2: SP 0.2: P 4.* Refers to I. L. Collier, C. C. May, C. C. More, R. G. Tyler.

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(10) **STONE, HENRY NORTON,** 1325 Nineteenth St. South, Birmingham, Ala. (Age 31. Born Worth, W. Va.) 1925 A. B., Cornell Univ. *TT 2: P 2.*—March 1925 to date with Virginia Bridge & Iron Co., until Dec. 1927 as Helper on fitting, riveting and templet making (over 7 months), Draftsman (1⅕ years) and Estimator and Designer (1 year); Jan. 1928-June 1929 Res. Engr. in charge of erection forces on James River Bridge, Newport News, Va., Mississippi River Bridge approaches, Vicksburg, Miss., Yazoo River Bridge, Redwood, Miss., and Southern R. R. Bridge at Richmond, Va.; since July 1929 Asst. Engr., about 2 years in Roanoke office designing erection schemes, planning work of Erection Dept., and having general supervision of field forces, and since May 1931 in charge of Birmingham Office of Erection Dept., preparing erection estimates, planning and supervising work of territory around Birmingham. *TT 6.1: SP 0.9: P 5.2: RC 2.3: D 0.9.*—*TT 8.1: SP 0.9: P 7.2: RC 2.3: D 0.9.* Refers to P. A. Blackwell, H. A. Davies, C. W. Ogden, P. V. Pennybacker, A. R. Peyton, W. N. Woodbury.

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(16) **STREET, RALPH WOOD,** 6444 Summit Street, Kansas City, Mo. (Age 49. Born St. Joseph, Mo.) 1905 LL.B., Law School, Univ. of Mich. *TT 2: P 2.*—April 1917 to Dec. 1922 Engr., Street & Co., Investment Banking Firm. *TT 5.7: P 5.7.*—Jan. 1923 to March 1924 with Kansas City Finance Co., in charge of design and construction of 14-story Land Bank Bldg., Kansas City, Mo. *TT 1.2: P 1.2: RC 1.2.*—Sept. 1923 to July 1929 conceived and had charge of development of Osage hydro-electric project for Missouri Hydro-Elec. Power Co. *TT 5.3: P 5.3: RC 5.3: D 2.*—Aug. 1929 to April 1930 had charge acquisition of reservoir and property claims of Osage Project for Union Elec. Light & Power Co. *TT 1.7: P 1.7: RC 1.7.*—April 1925 to date conceived and had charge of hydro-electric development of Current River for Missouri Hydro-Elec. Power Co. and Current River Power Co. and (since Oct. 1927) of Gasconade River for Gasconade River Power Co. *TT 7: P 7: RC 7: D 3.*—*TT 23: P 23: RC 15.2: D 5.* Refers to H. F. Anthony, H. C. Beckman, C. E. Brown, C. W. Brown, D. A. Dean, H. E. Riggs, J. E. S. Thorpe.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

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(11) **ANGILLY, CHARLES ENOCH, Jr.,** Assoc. M., 1152 West Seventy-fourth St., Los Angeles, Cal. (Elected June 6, 1927.) (Age 43. Born Providence, R. I.) 1911 B. S. in Civ. Eng., R. I. State Coll. *TT 4: P 4.*—Sept. 1917 to Feb. 1919 with U. S. Army, 4 months as 1st Sergeant, Co. B, 316th Engrs., 13 months 2d Lieut., Engrs. and 8 months with A. E. F., Div. of Constr. and Forestry, timber surveys, sawmill operation and road construction. *TT 1.2: SP 0.1: P 1.1: RC 1.1.*—April 1916 to Sept. 1917 and Feb. 1919 to date with City of Los Angeles, Bureau of Water Works and Supply, and (after Dec.

1928) Dept. of Water and Power (Water Div.), until Sept. 1917 in charge of field party, on surveys, San Fernando Valley Irrigation Dist., 6 months being in charge of two parties; Feb. to June 1919 Instrumentman and Chf. of Party on surveys; June 1919 to May 1925 on civil, mechanical and structural drafting, mapping, hydraulic computations, water-extension systems, water-works structures and buildings; May 1925 to Dec. 1928 in responsible charge of all activities of Drafting Div.; since Dec. 1928 Engr. of Design, in responsible charge of Design and Drafting Div. distribution system, reservoirs, dams and water-works structures. *TT 14.6: P 14.6: RC 10.1: D 8.5.*—*TT 19.8: SP 0.1: P 19.7: RC 10.1: D 8.5.* Refers to W. W. Hurlbut, H. L. Jacques, W. Mulholland, J. E. Phillips, O. A. Stone, H. A. Van Norman.

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(15) BELL, HOWARD FRED, Assoc. M., Cody, Wyo. (Elected Junior May 31, 1910; Assoc. M. April 18, 1916.) (Age 47. Born Windsor, Ohio.) 1909 C. E., Ohio State Univ. *TT 4: P 4.*—Sept. 1909 to May 1910 Asst. to County Engr., Ashtabula County, Ohio. *TT 0.6: SP 0.1: P 0.5: D 0.5.*—May 1910 to Sept. 1911 Asst. Engr. with B. F. Hewit, Engr. and Contr., Geneva, Ohio. *TT 1.3: P 1.3: RC 1.3: D 0.5.*—Dec. 1911 to date in private practice at Cody, Wyo., on irrigation, drainage and municipal engineering, involving water-works, sewers, etc., and general engineering work, being Engr. for ten drainage and seven irrigation districts in Wyoming, also County Surveyor for Park County (Jan. 1913 to date) and Engr. for Town of Cody (June 1913 to June 1915 and June 1921 to date); work has included surveys, specifications, plans, design and construction of drainage systems, design and construction of water-works systems, reservoirs dam and irrigation structures, construction of tunnel (1 130 ft.), canal construction and reconstruction, surveys, plans and construction of sewer systems, etc. (over $1 000 000); also surveys, plans, specifications, investigations, location, reports, estimates, etc. for organization of irrigation and paving districts, etc. (work given in detail in application). *TT 20: P 20: RC 20: D 15.*—*TT 25.9: SP 0.1: P 25.8: RC 21.3: D 16.* Refers to P. Beall, C. H. Bowman, H. D. Comstock, J. A. Elliott, E. C. Gwillim, W. S. Hanna, R. A. Hart, J. R. Iakisch, R. E. Richardson, Z. E. Sevison.

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(15) BOATNER, MARK MAYO, Jr., Assoc. M., 7739 Hampson St., New Orleans, La. (Elected Feb. 10, 1930.) (Age 35. Born New Orleans, La.) 1918 2d Lieut., U. S. Mil. Acad. *TT 4: P 4.*—Graduate, Basic and Civ. Eng. courses, Engr. School, U. S. Army.—June 1920 to Dec. 1929 with U. S. Army until Dec. 1920 as Co. Commander with Recruit Educational Center and 9th Engrs.; June 1921 to June 1922 at Post Headquarters, Ft. Humphreys, Va., Mess Officers, R. O. T. C., Post Exchange, Ordnance, Police and Prison, Recreation and C. W. S. Officers; July 1922 Instructor, Engr., Pennsylvania National Guard; Aug. 1922 to Jan. 1923 Co. Commander, Headquarters and Service Co., 13th Engrs., also on preparation of technical regulations; Jan. 1923 to Jan. 1926 Lieut. and Co. Commander 11th Engrs., Panama, including military survey of Panama, being in charge of detached camps or reconnaissance; March 1926 to Dec. 1929 Associate Prof. Mil. Dept., Iowa State Coll., Engr. Unit, engineering and military subjects. *TT 7.1: SP 0.4: P 6.7: RC 5.3: D 0.2.*—Jan. 1930 to date Asst. to Dist. Engr., U. S. Engr. Office, Second New Orleans Dist., in responsible charge of work, including various engineering design, being Chf., Engr. Repair Dept., repair, maintenance, valuation, design of plant, Chf., Eng. Div., approx. 12 000 sq. miles Alluvial Valley Map, including use of aerial pictures, technical reports, studies, design, drafting, permits, surveys, gauges and observations, and Chf., 2d Field Area and (at present) 3d Field Area, on levees, revetements and field operations for Red River below Alexandria, Mississippi River between Old River and Baton Rouge and entire Atchfalaya River, also in Lafourche, Pontchartrain, Lake Borgne and Barataria Levee Districts, including design and development of equipment for taking approx. 2 000 borings and probings in Atchafalaya Basin and of equipment for operation of Bonnet Carre Spillway. *TT 2.3: P 2.3: RC 2.3: D 2.3.*—*TT 13.4: SP 0.4: P 13: RC 7.6: D 2.5.* Refers to F. C. Carey, J. F. Coleman, A. H. Fuller, T. H. Jackson, A. Marston, H. D. Moore, P. S. Reinecke.

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(1) DAVIDSON, FRANK ALBERT, Assoc. M., 204 East Thirty-ninth St., New York City. (Elected Nov. 12. 1928.) Age 35. Born Brooklyn, N. Y.) Licensed Prof. Engr., New York State. 1929 C. E., Pol. Inst. of Brooklyn.—Nov. 1916 to July 1920 (except June to Dec. 1918 with U. S. Navy) with Harris Structural Steel Co., New York City, as Draftsman, on structural detailing, estimating, drafting, etc., and (4 months) Asst. to Supt. of erection for dirigible balloon hangar, Langley field, Va. *TT 1.5: SP 1.5.*—Aug. 1920 to May 1921 with Frederick Snare Corporation, New York City, drafting for contracting engineers, including detailing structural steel and reinforced concrete and standard-

izing equipment. *TT 0.5: SP 0.5.*—June 1921 to date with Edward Corning Co., New York City, 2 years as Draftsman, drafting, some design of structural members, checking and approving subcontractors' plans and details, including structural steel details, etc., 3 years Asst. Engr. in Chg. of Constr., making independent solutions of field problems, assisting in design and redesign of structural steel, supervising installation of reinforced concrete, and since June 1926 Supt. of Constr., having complete supervision of materials, workmanship and progress, designing in steel, concrete and wood, etc.; work has included charge of designing in connection with Peabody Home, Samaritan Home for the Aged, apartment house for Union Theological Seminary, alteration to residence for George Whitney, all in New York City, and alteration to residence and new stable for George Murname, Brookville, N. Y., etc. *TT 9.7: SP 1: P 8.7: RC 5.7: D 5.7.*—*TT 11.7: SP 3: P 8.7: RC 5.7: D 5.7.* Refers to A. N. Aeryns, H. R. Codwise, H. P. Hammond, C. W. Hudson, E. E Seelye, E. J. Squire.

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(1) **FOSTER, HENRY ALDEN,** Assoc. M., 266 Tichenor Ave., South Orange, N. J. (Elected April 3, 1922.) (Age 40. Born Geneva, N. Y.) 1913 B. S., Univ. of Ariz. 1916 C. E., Cornell Univ. *TT 4: P 4.*—Sept. 1913 to Sept. 1914 Draftsman and Instrumentman, Interborough Rapid Transit Co., New York City, 6 months drafting track-work details and maintenance and 6 months leveling and fieldwork, reconstruction of 3d Ave. Elec. R. R. *TT 0.5: SP 0.5.*—Oct. 1916 to Sept. 1917 Structural Detailer, Hay Foundry & Iron Works, Newark, N. J., details of steel framing, office and mill buildings, power houses. *TT 0.5: SP 0.5.*—Sept. 1917 to July 1919 with U. S. Army as Private, etc., to 2d Lieut., 1⅔ years in France; technical work with 29th Engrs. (topographical), fieldwork in triangulation, computation of triangulation and traverse, considerable geodetic and astronomical work. *TT 1.5: P 1.5: RC 1.*—Sept. 1919 to Feb. 1921 and Nov. 1922 to Sept. 1925 with Dwight P. Robinson & Co., New York City, until Feb. 1921 as Structural Designer, designing and checking design of large steam power plants and industrial buildings, mostly structural steelwork, and after Nov. 1922 Structural Engr., designing and checking structural steel and Office Engr. on building construction, contracts, specifications, approving contractors' shop details, etc. *TT 4.3: P 4.3: RC 3.8: D 1.*—March 1921 to Nov. 1922 and Sept. 1925 to date with Parsons, Klapp, Brinckerhoff & Douglas, New York City, until Nov. 1922 as Asst. Engr. on New York water power investigation, investigating hydrology of New York State streams, studies of rainfall, stream flow, storage, power capacity, application of theory of probability to runoff studies, and since Sept. 1925 Asst. to Chf. Engr. designing and supervising design of hydro-electric plants, arch bridges, buildings, foundations, dams, investigations and reports, contracts and specifications. *TT 8.2: P 8.2: RC 8.2: D 6.*—*TT 19: SP 1: P 18: RC 13: D 7.* Refers to W. E. Belcher, W. K. Brownell, W. J. Douglas, E. E. Halmos, J. P. Hogan, C. E. Sudler, F. S. Tainter.

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(10) **HALL, WARREN ESTERLY,** Assoc. M., 701 Peters Bldg., Atlanta, Ga. (Elected Junior Nov. 5, 1907; Assoc. M. June 30, 1910.) (Age 50. Born Dawsonville, Ga.) Sept. 1897 to Feb. 1899 and Feb. 1900 to June 1902 student, Georgia School of Technology. *TT 2: P 2.*—Feb. 1899 to Jan. 1900, June 1902 to Dec. 1903 and Jan. to June 1912 with Hall Bros., Civ., Min. and Hydr. Engrs., Atlanta, Ga., as Rodman, Instrumentman in charge of field parties, and (after Jan. 1912) on surveys, design and construction, being Res. Engr. on water-works construction, Rhinehart Coll., Canton, Ga. *TT 2.1: SP 0.5: P 1.6: RC 0.2: D 0.1.*—Jan. 1904 to Sept. 1908, June 1912 to Sept. 1917 and Aug. 1919 to Dec. 1923 with Water Resources Branch, U. S. Geological Survey, Southeastern and Gulf States Dists., until Sept. 1908 as Field Asst., Jun. Engr. and Asst. Engr., and remainder of time Dist. Engr. in charge on stream gauging, river surveys, installing gauges and gauging equipment, computing,, compiling and editing stream-flow data, etc. *TT 14.5: P 14.5: RC 9.6: D 1.*—Oct. 1908 to May 1911 with Porto Rico Irrigation Service as Asst. Engr. on surveys and construction of canals, dams and tunnels, and Res. Engr., Patillas Canal, Guayabel Dam (investigations) and El Toro Negro Dam and Tunnel. *TT 2.7: P 2.7: RC 2.7.*—June–Dec. 1911 Contrs. Engr., B. H. Hardaway Constr. Co., Columbus, Ga., on construction of Tallulah Falls dam and power house foundations. *TT 0.7: P 0.7.*—Oct. 1917 to July 1919 with U. S. Army, 2 months at Officers' Training Camp, Dec. 1917 to July 1918 Capt., Co. D, 506th Engrs., building railroads and docks, Aug. 1918 to Feb. 1919 Asst. Water-Supply Engr., and Water-Supply Engr., Base Sec. No. 2, after March 1919 in hospital; from Jan. 1918 to March 1919 with A. E. F., France. *TT 1.8: P 1.8: RC 1.8 D 0.4.*—Jan. 1924 to Jan. 1925 Secy. and Treas., Western North Carolina, Inc., gathering engineering and resources

statistics. *TT 0.6: SP 0.6.*—Feb. 1925 to Oct. 1926 with Chas. E. Waddell & Co., Civ. Engrs., Asheville, N. C., on surveys, design and construction, from May 1925 to Feb. 1926 being Res. Engr., Bee Tree Dam (hydraulic fill earth dam, 175 ft. high). *TT 1.8: P 1.8: RC 0.8: D 0.2.*—Nov. 1926 to date with B. M. Hall & Sons, until Dec. 1928 as Field and Designing Engr., and since Jan. 1929 member of firm, general engineering, including design and construction of earth dams, water supply and sewerage, and Consultants on water-power litigation. *TT 5.4: P 5.4: RC 5.4: D 2.*—*TT 31.6: SP 1.1: P 30.5: RC 20.5: D 3.7.* Refers to B. S. Drane, N. C. Grover, B. M. Hall, Jr., M. R. Hall, J. H. Johnston, L. A. Kibbe, W. R. King, W. A. Lamb, H. D. McGlashan, C. G. Paulsen, C. H. Pierce, T. Saville, M. R. Scharff, C. E. Waddell, C. C. Whitaker.

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(1) LINEBERGER, WALTER FRANKLIN, Assoc. M., 3651 Pacific Ave., Long Beach, Cal. (Elected May 4, 1909.) (Age 51. Born Bolivar, Tenn.) Sept. 1898 to Feb. 1901 special student in Civ. Eng., Rensselaer Polytechnic Inst.—Summers 1899 and 1900 Rodman, Chainman, Axeman, Leveler, etc., with City Engr. and County Surveyor, Los Angeles.—Sept. 1902 to May 1903 Track Engr. and Asst. Res. Engr., National R. R. of Mexico on masonry and road-bed construction. *TT 0.7: P 0.7: RC 0.7: D 0.3.*—May to Sept. 1903 Asst. Res. Engr., National R. R. of Tehauntepec at Rincon Antonio, on track-work, masonry construction and structural steel drafting. *TT 0.2: P 0.2: D 0.2.*—Sept. 1903 to Jan. 1904 Draftsman and Surveyor with S. Pearson & Sons, Ltd., on harbor work. *TT 0.4: P 0.4: RC 0.4: D 0.2.*—Feb. to Aug. 1904 Engr. and Draftsman, Mexican Light & Power Co., Necaxa hydro-electric plant, and on location of transmission line, Necaxa to Mexico City. *TT 0.6: P 0.6: RC 0.6.*—Sept. 1904 to Jan. 1906 First Asst. to Chf. Engr., Compania Agricola del Tlahualilo, at Lerdo and Tlahualilo, Mexico, in charge of masonry dam construction, hydrographic stream measurements, head-gate design, surveys, etc. *TT 1.4: P 1.4: RC 1.4.*—Jan. 1906 to Jan. 1907 Engr., Velardena (Mex.) Mining & Smelting Co., in charge of location and construction of smelter extension of Velardena Ry., on aerial tramway design, location and construction, storage reservoir construction, mine surveying, mapping, etc. *TT 1: P 1: RC 1: D 0.5.*—Jan. 1907 to Oct. 1909 member of firm, Lineberger & Rone, Civ. and Min. Engrs., Torreon, Coahuila, Mexico, on design and directing construction of irrigation and drainage systems, hydraulic and railroad construction, precise surveying, etc.; made precise measurement map of Torreon, investigation and report on irrigation methods throughout west for State Govt., etc. *TT 2.7: P 2.7: RC 2.7: D 1.*—Oct. 1909 to July 1917 Civ. Engr., in Southern California, harbor and dock, design and construction, and Chairman, Harbor Comm., Long Beach, Cal., involving administration of harbor and harbor improvements (about $5 000 000), docks, industrial developments and dredging; laid out and built several irrigation systems in Imperial Valley, hydrographic measurements, headgate designs, etc. *TT 7.7: SP 1: P 6.7: RC 6.7: D 1.*—July 1917 to Sept. 1919 Capt. of Engrs. (discharged as Major), U. S. Army, with A. E. F. in France, military engineering, sapping, etc. *TT 2.2: P 2.2: RC 2.2.*—Jan. 1920 to March 1921 Cons. Engr., Long Beach, on port work and irrigation. *TT 1: SP 0.2: P 0.8: RC 0.8: D 0.3.*—March 1921 to 1927 member of Congress, 9th Cal. Dist., being member of various committees, involving investigation, reports, etc., on river and harbor projects, waterways, bridges, shipping and shipping board problems, etc.; was consulted on many technical and administrative questions pertaining to government affairs. *TT 3: SP 3.*—March 1927 to March 1929 Gen. Mgr., of port developments and Cons. Engr. at Fort Lauderdale, Fla., on port construction (involving $5 000 000). *TT 2: P 2: RC 2: D 0.5.*—March to Dec. 1929 special engineering and executive work for Arundel Corporation, Baltimore, Md., Dredging and Gen. Constr. Contrs. *TT 0.4: SP 0.2: P 0.2: RC 0.2.*—Jan. 1930 to date Asst. to Pres., Standard Dredging Co., New York City (contracting), general port works construction and dredging operations throughout the United States, chiefly executive work and construction operations. *TT 1.7: SP 0.5: P 1.2: RC 1.2.*—*TT 25.3: SP 5: P 20.3: RC 20.1: D 4.2.* Refers to W. J. Barden, W. K. Barnard, H. L. Cooper, H. Deakyne, A. A. Fries, G. B. Hills, C. T. Leeds, E. Mead, G. W. Sackett, J. A. L. Waddell.

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(6) SHUBIN, SYDNEY ABRAM, Assoc. M., 998 Biltmore Ave., Dormont, Pittsburgh, Pa. (Elected June 7, 1926.) (Age 41. Born New York City.) Registered Prof. Engr., State of Pennsylvania. 1915 B. S. in Civ. Eng., and 1926 C. E., Cooper Union. *TT 4: P 4.*—May 1915 to Feb. 1916 with Buckingham Steel Co., Brooklyn, N. Y, drafting, designing and estimating structural steel *TT 0.6: SP 0.2: P 0.4: RC 0.1: D 0.3.*—March 1916 to Jan. 1917 with Lackawanna Steel Co., Buffalo, N. Y., on design and details for extension and maintenance of plants, traveling docks, loaders and unloaders. *TT 0.5: SP 0.3: P 0.2:*

D 0.2.—Feb. to Oct. 1917 Asst. Chf. Draftsman, Bollinger-Andrews Constr. Co., Verona, Pa., designing and checking; designed long-span steel trusses for large steam pipes, and had charge of design for new forge and foundry shop for Pittsburgh Steel Products Co. *TT 0.7: SP 0.1: P 0.6: RC 0.6: D 0.6.*—July 1918 to Aug. 1921 with Heyl & Patterson, Pittsburgh, Pa., checking, designing and estimating coal tipples, conveyors, etc., for coke and coal handling; designed coal-storage buildings (ten 360-ton coal bunkers). *TT 2.2: SP 0.7: P 1.5: RC 0.3: D 1.5.*—Nov. 1917 to June 1918 and Dec. 1921 to June 1922 with McClintic-Marshall Co., Pittsburgh, checking and designing details and connections on long-span railroad and highway bridges and buildings. *TT 1: SP 0.4: P 0.6: D 0.6.*—Sept. to Dec. 1921 with Pennsylvania State Highway Dept., checking, final estimates on concrete highway bridges and culverts. *TT 0.3: P 0.3: RC 0.3.*—July 1922 to March 1924 with J. Harold Rapp Co., Pittsburgh, checking, laying out work on industrial plants, office buildings and theatres; checked steel and erection plans for a theatre and market house for City of Memphis, Tenn. *TT 1.4: SP 0.4: P 1: RC 0.4: D 0.6.*—April 1924 to date Asst. Design Engr., Dept. of Public Works, Bureau of Bridges, Div. of Design, Allegheny County, Pa., designed, made plans, etc., for abutments, superstructures, approaches, caisson walls, piers, connections, etc., for various bridges over Monongahela and Allegheny Rivers; designed and had responsible charge of plans for Chartiers Creek Steel Truss and Girder Viaduct connecting Lincoln and William Penn Highways and for Pittsburgh approach (about 1 000 ft.) for 31st St. Bridge over Allegheny River; designed steel for basement, first and second floors and roof of Allegheny County Office Bldg.; made preliminary studies and calculations for South Tenth St. Bridge over Monogahela River; designed concrete gravity anchorages and river piers, made preliminary design for entire superstructure and had esponsible charge of design plans for super and substructures (work given in detail in application). *TT 7.8: SP 0.2: P 7.6: RC 6: D 7.5.*—*TT 18.5: SP 2.3: P 16.2: RC 7.7: D 11.3.* Refers to H. G. Appel, V. R. Covell, H. K. Dodge, C. K. Harvey, J. C. Jordan, R. S. Quick, T. J. Wilkerson.

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(6) **SMITH, CHESTER WASON,** Assoc. M., U. S. Engr. Office, Huntington, W. Va. (Elected Oct. 5, 1904.) (Age 66. Born Hudson, N. H.) April 1888 to June 1892 Rodman, Transitman and Head of Party with Aspinwall & Lincoln, Boston, on surveys and highway construction. *TT 4.2: P 4.2.*—June 1892 to July 1895 Head of Party on sewer and water-works construction, surveys, estimates, etc., until Aug. 1893 with City of Marlboro, Mass., then with Massachusetts State Board of Health. *TT 3.2: P 3.2.*—Aug. 1895 to Dec. 1904 with Boston Metropolitan Water Board, on surveys and estimates, Asst. Engr. on construction of brick and pipe sewer, filter beds, concrete reservoir, and temporary and protective works for Wachusett Dam, Div. Engr. on construction of, and (1904) also Chf. Inspector on the dam. *TT 9.4: P 9.4: RC 4.*—Dec. 1904 to Nov. 1910 Constr. Engr., Dept. of Interior, U. S. Reclamation Service, in charge of construction of Roosevelt Dam and allied works, canals, tunnels, tunnel lining, reinforced concrete conduit, bridges, etc. *TT 5.9: P 5.9: RC 5.9.*—Nov. 1910 to Jan. 1911 Engr. in charge of construction of private irrigation dam, New Mexico. *TT 0.2: P 0.2: RC 0.2.*—Jan. to March 1911 made estimate and report on dam and irrigation project in Medina, Tex., for John R. Freeman. *TT 0.2: P 0.2: RC 0.2.*—April 1911 to March 1912 surveys and estimates for Salmon River hydro-electric power project, for Ontario Power Co., and Nov. 1912 to July 1913 on investigations and estimates of dam for hydro-electric power project on Kootenai River, Mont., and canal and dam studies for project on Platte River, both for Westinghouse, Church, Kerr & Co. *TT 1.5: P 1.5: RC 0.5.*—Sept. to Dec. 1913 with John W. Young, made preliminary plans and estimates, also operating estimates for rail water terminal, New York Harbor. *TT 0.4: P 0.4: RC 0.4: D 0.4.*—April to Oct. 1914 with MacArthur Bros. Co. in Peru, studying irrigation, and making preliminary designs and estimates for five or six irrigation pojects. *TT 0.5: P 0.5: RC 0.5: D 0.1.*—1914 to 1916 (short periods) Cons. Engr., New York City, made plans and estimates for rail water terminal, San Francisco Bay, and for shipyard, New York Bay (Staten Island) and plans for repairs to several masonry dams, Scranton, Pa. *TT 0.5: P 0.5: RC 0.5: D 0.3.*—Sept. 1916 to Nov. 1918 with L. H. Shattuck Co., Manchester, N. H., until April 1917 on preliminary studies and estimates for several small water powers, then Engr. of wooden shipyard, design and construction of plant, production programs, cost studies. *TT 2.3: P 2.3: D 0.2.*—Nov. 1918 to May 1919 Engr., Production Dept. of steel shipyard at Portsmouth, production programs, cost studies, etc. *TT 0.5: P 0.5.*—Aug 1919 to March 1920 Engr. for a sugar estate in Santo Domingo, on surveys, plans and some construction, irrigation system. *TT 0.7: P 0.7: RC 0.7: D 0.3.*—May 1920 to Jan. 1922 Res. Engr., Connecticut State Highway Dept., on construction of bridge (7 spans) across Con-

necticut River at Windsor Locks, Conn. *TT 1.6: P 1.6: RC 1.6.*—April 1922 to April 1924 Engr. for an American Syndicate, on harbor work for Port of Palermo, Sicily, and (4 months) on Sicilian narrow-gauge railway construction. *TT 2: P 2: RC 2: D 0.4.*—April to Sept. 1924 with a Greek-American Syndicate in Athens, Greece, on housing projects and estimates, review of plans and estimates for land reclamation projects, Thessaly and Salonica, and (4 months) on designs and estimates for additional domestic water supply for Athens. *TT 0.5: P 0.5: RC 0.5: D 0.3.*—Oct. to Dec. 1924 Cons. Engr., Rochester (N.Y.) Gas & Elec. Corporation, on design and estimates for Mt. Morris Dam. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—April to Dec. 1925 with Aspinwall & Lincoln, Boston, as Chf. of Survey Party. *TT. 0.7: P 0.7: RC 0.7.*—Dec. 1925 to Oct. 1926 Cons. Engr., J. G. White Eng. Co., on irrigation works for Mexican Govt., surveys, preliminary plans and estimates for irrigation system, Gueraro, Chih., Mexico. *TT 0.8: P 0.8: RC 0.8: D 0.3.*—April to May 1927 Cons. Engr., International Paper Co., on appraisal of hydro-electric plant (200 000 h. p.) in Newfoundland, also estimates of stream flow and power possibilities on adjacent watersheds. *TT 0.2: P 0.2: RC 0.2.*—Oct. 1927 to Feb. 1928 Engr., J. G. White Eng. Co., on surveys, examinations, estimates and report on hydro-electric plant in southern Chile. *TT 0.3: P 0.3: RC 0.3.*—Feb. to April 1928 Engr., Elec. Bond & Share Co., New York City, made examination, estimate and report on hydro-electric project in Colombia. *TT 0.2: P 0.2: RC 0.2.*—May 1928 to Nov. 1929 Res. Engr., Vermont State Highway Comm., Montpelier, Vt., on construction of three bridges, some road work. *TT 1.5: P 1.5: RC 1.5.*—Nov. 1930 to date with War Dept., U. S. Engr. Office, Huntington, W. Va., until March 1931 on preliminary plans and estimates for two locks and dams on Kanawha River, and since then Operations Engr. in charge of outside operations, Huntington Dist.—*TT 1.3: P 1.3: RC 1.3: D 0.2.*—*TT 38.8: P 38.8: RC 22.2: D 2.9.* Refers to A. S. Crane, A. P. Davis, F. H. Fay, J. R. Freeman, X. H. Goodnough, F. H. Newell, G. G. Shedd, F. E. Winsor.

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(15) WILLIFORD, CARL LEX, Assoc. M., 320 Records Bldg., Dallas, Tex. (Elected March 7, 1921.) (Age 43. Born Walnut Springs, Tex.) 1911 B. S., Tex. Agri. & Mech. Coll. *TT 4: P 4.*—Summers 1907 and 1908 and Jan. 1909 to Jan. 1910 with Texas Central R. R., as Chainman, Rodman and Instrumentman. *TT 0.7: SP 0.2: P 0.5.*—April to Sept. 1912 Draftsman and Instrumentman, Eng. Dept., City of Waco, Tex., street grades, etc. *TT 0.3: SP 0.1: P 0.2.*—Sept. 1912 to Sept. 1917 Draftsman, Instrumentman and Asst. Engr., City of Houston, Tex., sewers and sewage-disposal design and construction. *TT 4: SP 1: P 3: RC 3: D 3.*—Dec. 1917 to Feb. 1919 with U. S. Army, Air Service, as Personnel Officer, Asst. Engr. Officer at Ellington Field, Houston, Tex., and Engr. Officer on water supply and sewage disposal, Air Service Headquarters, Washington, D. C. *TT 1.1: P 1.1: RC 1.1: D 0.2.*—March 1919 to May 1921 Engr., Refining Dept., The Texas Co., Houston, Tex., assisted on design for high-pressure gasoline stills, including reinforced concrete and structural steel design, machinery and piping layouts, etc., revised standard designs, made original designs of power houses, pumping plants, buildings, machinery layouts, etc. *TT 1.5: SP 0.5: P 1: D 1.*—May 1921 to May 1925 Office Engr., Nagle, Witt, Rollins Eng. Co., Cons. Engrs., Dallas, Tex., highway design and construction, etc., being responsible for detail plans for over 300 miles of highways; work involved laying grade lines, planning roadway sections, design of structures for railroad grade separations, etc., detail plans for pavements, etc.; was also responsible for design and construction of private and municipal work, including water-works, sewerage and sewage-disposal plant for Garland, Tex. (about $100 000), etc. *TT 4: P 4: RC: D 4.*—May 1925 to 1926 Office Mgr., Austin Bridge Co., Dallas, Tex., bridge contracting, being Asst. to Pres., some bridge design, estimating, etc. *TT 0.7: P 0.7.*—Feb. 1926 to June 1928 Engr. and member of firm, Nagle, Witt, Rollins Eng. Co., Dallas and Houston, Tex., general consulting practice, 1 year in charge of branch office at Houston, and of municipal street paving at Goose Creek, Tex., and of plans for storm drainage and street paving at Port Arthur, Tex., then assisted on plans for an irrigation project in Nuecas County (not built) and plans and report for smaller project in Frio County, Tex., and Res. Engr. on plans and construction of 16 miles of state and county highways in Limestone County, etc. *TT 2.3: P 2.3: RC 2.3: D 2.3.*—June 1928 to date Res. Engr., State Highway Dept., Dallas, Tex., on design and construction of highways, having complete charge of preliminary surveys, design and construction of state highway (120 miles) in six counties. *TT 3.7: P 3.7: RC 3.5: D 3.5.*—*TT 22.3: SP 1.8: P 20.5: RC 13.8: D 14.* Refers to G. Gilchrist, J. T. L. McNew, J. C. McVea, J. M. Nagle, A. P. Rollins, A. D. Stivers, R. A. Thompson.

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(1) **WILSON, PERCY SUYDAM,** Assoc. M., 325 Washington St., Glen Ridge, N. J. (Elected Junior Sept. 12, 1921; Assoc. M. Dec. 15, 1924.) (Age 35. Born Montclair, N. J.) 1918 C. E., and 1920 M. C. E., Cornell Univ. *TT 4: P 4.*—Summer 1918 Asst. Engr., Cummings Structural Concrete Co. of Pittsburgh, Pa., on concrete barge construction.—Summer 1919 Instructor, Summer Survey Camp, Cornell Univ.—March to Dec. 1920 Draftsman and Asst. Engr., Eng. Dept., National Aniline & Chemical Co., Buffalo, N. Y., drafting, inspecting and surveying for plant maintenance and construction. *TT 0.5: SP 0.3: P 0.2.*—Jan. 1921 to June 1925 Asst. Engr. with James H. Fuertes, Cons. Engr., designing and superintending construction of water supply and sewerage works, including enlargement of filter plant and pump station, new force mains and 30,000 000-gal. covered reservoir at Harrisburg, Pa., new 64 000 000-gal. filter plant, with intake, screen plant, aerator and clear well at Denver, Colo. *TT 4: SP 0.4: P 3.6: RC 3.1: D 1.9.*—Nov. 1925 to May 1926 with Spencer, White & Prentis, Contrs., as Engr. and Constr. Supt. on heavy building foundations in New York City, piling, unedrpinning, etc. *TT 0.6: P 0.6: RC 0.5: D 0.1.*—June 1926 to Aug. 1927 Chf. Engr. and Supt., New Rochelle (N. Y.) Water Co., on engineering in connection with operation and construction including extensions of distributing systems, land holdings, etc.; as Supt. was also responsible for executive management. *TT 1.2: P 1.2: RC 1.2: D 0.7.*—Sept. 1927 to date Supt. of Operation, Community Water Service Co., New York City, in charge of all affiliated companies (about 40) and of engineering in connection with operation and construction, including design and construction of water-works plants and equipment; also executive work. *TT 4.5: P 4.5: RC 4.5: D 1.5.*—*TT 14.9: SP 0.8: P 14.1: RC 9.3: D 4.2.* Refers to R. K. Blanchard, R. H. Gould, C. Haydock, R. J. Newsom, A. V. Ruggles, J. A. Wade, F. H. Weed, E. K. Wilson.

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(10) **WINCHESTER, THOMAS HARRISON,** Assoc. M., 119 Vista Circle, Macon, Ga. (Elected Junior, May 4, 1909; Assoc. M., Nov. 12, 1913.) (Age 47. Born Macon, Ga.) Student, Va. Mil. Inst. (Sept. 1902 to Nov. 1904) and Univ. of Ga. (Feb. 1905 to June 1906). *TT 1.5: P 1.5.*—Prior to Nov. 1908 (about 2 years) on railroad and reservoir surveys. *TT 1.2: SP 0.8: P 04: RC 0.4.*—Nov. 1908 to Feb. 1909 with Central Georgia Power Co., Macon, in charge of party on surveys for power development. *TT 0.1: P 0.1: RC 0 1.*—Feb. 1909 to June 1911 Instrumentman, J. G. White & Co., New York City. *TT 1.7: SP 0.7: P 1: RC 1.*—July 1911 to July 1913 Engr., Stone & Webster Eng. Corporation, Boston, Mass., until March 1913 on construction of a 40 000-h.p. hydro-electric plant for Columbus Power Co. (about $2 000 000), then on construction of two substations and two distribution lines (total about $30 000). *TT 2.1: P 2 1: RC 2.1.*—Aug. to Nov. 1913 with Juliette Milling Co., Macon, Ga., in charge of survey for hydro plant on Ocmulgee River. *TT 0.3: P 0.3: RC 0.3.*—Nov. 1913 to July 1915 member of firm, Winchester & King, Columbus, Ga., on surveys, highway location, etc., plans for improvements to plant of Columbus Water Works, etc. *TT 1.2: SP 0.5: P 0.7: RC 0.5: D 0.2.*—July 1915 to Jan. 1917 Engr., Central Georgia Power Co., Macon, in charge of location and construction of 16 miles of 66-kv. transmission line and of installation of two 3 000-kw. generators and water turbines, also on investigation for an additional hydro-electrical plant. *TT 1.5: P 1.5: RC 1.4: D 0.1.*—Jan. to May 1917 Engr. Hardaway Contr. Co., Columbus, on location and construction of 10 miles of railroad at Lugoff, S. C. *TT 0.4: P 0.4: RC 0.4.*—May to Dec. 1917 with Stone & Webster Eng. Corporation, Boston, Mass., 3 months as Engr. on improvements at hydro plant near Columbus, Ga., 3 months Gen. Foreman on construction of concrete coal bunker at Youngstown, Ohio and one month Engr., in charge of first group of ways at Hog Island, Pa. *TT 0.5: P 0.5: RC 0.5.*—Aug. 1918 to Jan. 1919 1st Lieut., U. S. Army. *TT 0.2: SP 0.2.*—Dec. 1917 to Aug. 1918 and Feb. 1919 to Aug. 1924 with Rust Eng. Co., Pittsburgh, Pa., about 8 months supervising small jobs, about five months Estimator in Eng. Dep., about 9 months Gen. Foreman, in charge of constructing concrete reservoir and earth dam for water supply at Bucyrus, Ohio, about 8 months Asst. to Gen. Foreman at Norfolk, Va., about one year in charge of brick plant, about 8 months Asst. to Supt. on various construction and about 2 years Engr., Chimney Dept., designing and supervising construction of brick and concrete chimneys. *TT 5.2: SP 1.1: P 4.1: RC 3.1: D 1.*—Aug. 1924 to date Engr., Central Georgia Power Co., Macon, Ga. (now Georgia Power Co.), about 4 years in charge of surveys, including approx. 160 miles of transmission lines and about 75 000 acres of land, about 1½ years supervising operation of Macon St. Ry., about 2 years in charge of surveying, about 2 years on miscellaneous reports and investigations, etc., on streams, including surveys to show effect of hydro-plant operation on certain lands, on design of automatic dumping flashboards (5 ft. high) on top of two dams, and of four

small dams (two timber, one reinforced concrete and one masonry). *TT 7.6: P 7.6: RC 6.6: D 1.*—*TT 23.5: SP 3.3: P 20.2: RC 16.4: D 2.3.* Refers to W. D. Hull, R. F. Rhodes, H. B. Rust, C. A. Smith, G. R. Solomon, C. M. Strahan, W. H. Taylor, 3d.

FROM THE GRADE OF JUNIOR

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(1) AYERS, WILLIAM DeWITT, Jun., 40 Chapman Pl., Bayshore, N. Y. (Elected Oct. 1, 1926.) (Age 29. Born Branchville, N. J.) Prof. Engr. and Land Surveyor, New York State. 1925 C. E., Lehigh Univ. *TT 4: P 4.*—Summer 1923 Rodman and Chainman, County Engr.'s Office, Sussex County, N. J.—March to June 1925 Transitman and Asst. Chf. of Party with Edmund R. Halsey, Newark, N. J. *TT 0.3: P 0.3.*—June to Oct. 1925 Material Clerk and Timekeeper, Turner Constr. Co., New York City. *TT 0.3: P 0.3.*—Oct. 1925 to March 1927 with Wallace H. Halsey, Southampton, N. Y., until Jan. 1926 as Asst. Res. Engr. on title survey (approx. 9 000 acres), Montauk, N. Y., then (6 months) Engr. checking about 6 000 acres of the tract, calculating surveys and subdivisions, surveys, one subdivision layout, five sewage-disposal plants (private homes), in charge of two complete layouts, and after July 1926 Res. Engr. on title and triangulation survey (about 2 500 acres), soundings in Barnegat Bay, N. J. (over 3 500 acres), preliminary road survey (2 miles) and planetable survey (over 250 acres). *TT 1.5: P 1.5: RC 0.8.*—March to July 1927 on city planning for Henry Phipps Estate under Mr. Frost of Bennett, Parsons & Frost, Archts. and Engrs., Chicago, Ill. *TT 0.3: P 0.3: D 0.2.*—Sept. 1927 to May 1931 in private practice, 2 years as partner of W. H. Halsey, then alone, on various surveys, including preliminary surveys for roads, sewers, curbs, gutters, etc., topographical survey of Beach Dunes (300 acres), etc., preliminary work for sewers, field work, plans, etc. for five dams and ponds, design for typioal dam and pond; engineering work on streets, curbs and gutters, sidewalks, drainage, subdivisions, dredging, etc.; prepared plans and specifications and supervised construction of road (6½ miles, gravel, oiled surface), prepared preliminary plans for additional road (3½ miles), prepared various engineering reports, ocean-front roads, a new village, including layout, streets, paving cost, bulkheading, design and cost estimate of water system, etc. *TT 3.6: P 3.6: RC 3.6: D 1.*—May 1931 to date with Wallace H. Halsey, C. E., Inc., in responsible charge of engineering dredging project (18 miles), etc., being Mgr., Islip office. *TT 0.9: P 0.9: RC 0.9.*—*TT 10.9: P 10.9: RC 5.3: D 1.2.* Refers to S. A. Becker, M. O. Fuller, V. Gelineau, W. H. Halsey, H. G. Payrow, E. H. Uhler, E. W. Wolf.

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(1) BURKE, WALTER ANTHONY, Jr., Jun., 1220 Park Ave., New York City. (Elected Oct. 14, 1929.) (Age 28. Born New York City.) 1927 B. S., Sheffield Sci. School, Yale Univ. *TT 4: P 4.*—Oct. 1927 to Jan. 1928 Estimator, Erection Co. of New York. *TT 0.2: SP 0.1: P 0.1.*—Jan. 1928 to date with Thos. Crimmins Contr. Co., New York City, until May 1928 as Transitman laying out foundation work, May to July 1928 Estimator, July to Nov. 1928 Field Engr. supervising excavation, Nov. 1928 to June 1929 Office Asst. Engr., construction equipment investigation and (after April 1929) estimating, June 1929 to Jan. 1930 Field Supt. in charge of building foundation construction and (after Oct. 1929) of construction equipment, Jan. 1930 to Feb. 1931 Field Engr., and since Feb. 1931 Treas., on foundation work sheeting and underpinning design, and (after June 1930) estimating, planning and supervising. *TT 3.9: SP 0.3: P 3.6: RC 3: D 0.3.*—*TT 8.1: SP 0.4: P 7.7: RC 3: D 0.3.* Refers to F. N. Benedict, C. T. Bishop, E. W. Borough, A. H. Jorgensen, C. S. Landers, F. A. Snyder, W. H. J. Vollmer.

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(15) FRANKLIN, WILLIAM ROBERT, Jun., Brownsville, Tex. (Elected Oct. 1, 1926.) (Age 27. Born New York City.) 1926 S. B., Mass. Inst. Tech. *TT 4: P 4.*—June 1926 to Aug. 1927 Constr. Supt., Archt. and Eng. Dept., S. S. Kresge Co., Detroit, Mich., 6 months on maintenance and repair to store buildings and 6 months Archt.'s Supt. on buildings under construction. *TT 0.8: SP 0.3: P 0.5: RC 0.5.*—Sept. 1927 to Aug. 1929 Gen. Supt., Constr. Dept., McCrory Stores Corporation, New York City, in charge of field work, including maintenance and repairs, surveys, appraisal of buildings, adjustment of fire losses, Archt.'s Supt. on buildings under construction, surveys, reports and recommendations for alterations to stores. *TT 2: P 2: RC 2.*—Aug. 1929 to date co-partner with W. A. Velten Constr. Co., Brownsville, Tex., being Office Mgr., estimating and purchasing, and Field Supt. on sales and designs. *TT 2.6: P 2.6: RC 2.6: D 0.2.*—*TT 9.4: SP 0.3: P 9.1: RC 5.1: D 0.2.* Refers to J. B. Babcock, 3d., C. B. Breed, O. W. Freeman, B. S. Merrill, C. H. Sutherland.

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(7) **KROENING, WALTER EDWIN,** Jun., 1517 North Twenty-eighth St., Milwaukee, Wis. (Elected June 10, 1929.) (Age 29. Born Milwaukee, Wis.) June 1921 to date with Sewerage Comm., Milwaukee, Wis., 5 years as Draftsman on sewers, disposal plant, etc., 4 years Designer, designing and checking sewers, sewage-booster stations, etc., in Milwaukee Metropolitan Dist., population forecasts and sewage-flow investigations, and since June 1930 Senior Engr., in charge of design and plans for all main and intercepting sewers and sewage-pumping stations, checking sewer designs, cooperating with Chf. Chemist on industrial-waste pretreatment, engineering reports, etc. *TT 8.3: SP 2.5: P 5.8: RC 1.8: D 1.8.* Refers to J. L. Ferebee, T. C. Hatton, H. R. Holmes, W. Landsiedel, H. E. Nicol, D. W. Townsend, F. W. Ullius, H. C. Webster.

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(1) **LOESER, JESSE JOHN,** Jun., 95 Tuers Ave., Jersey City, N. J. (Elected July 12, 1926.) (Age 32. Born New York City.) 1924 B. S. in Civ. Eng., New York Univ. *TT 4: P 4.*—July to Dec. 1928 Engr. Miller Daybill & Co., New York City, in charge of lines and grades for foundations of Western Union Telegraph Bldg.—*TT 0.5: P 0.5.*—June 1924 to July 1928 and Dec. 1928 to Sept. 1931 with Turner Constr. Co., New York City, until June 1926 as Timekeeper, Material Clerk and Job Accountant; June 1926 to Dec. 1927 Engr. in charge of lines and grades on four buildings; Dec. 1927 to July 1928 and Dec. 1928 to Dec. 1930 Asst. Supt. in charge of reinforcing steel on four buildings ($1 000 000 to $1 500 000 and of plans and specifications and finishing trades on Ford Motor Co. job at Edgewater, N. J., and of interior masonry work on 5 buildings at Pilgrim State Hospital, Brentwood, L. I., and Dec. 1930 to Sept. 1931 Supt. in charge of finshing trades and of completing these buildings. *TT 5.9: SP 1: P 4.9: RC 2.8.*—Dec. 1931 to Feb. 1932 Supt., C & W Constr. Co., New York City, in charge of completing Public School 106, Bronx, *TT 0.2: P 0.2: RC 0.2.*—*TT 10.6: SP 1: P 9.6: RC 3.* Refers to DeF. H. Dixon, E. G. Hooper, W. W. Roberts, Jr., C. T. Schwarze, T. A. Smith, C. H. Snow, A. C. Tozzer, D. S. Trowbridge, H. C. Turner.

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(14) **MORRISS, BENHAM EPES,** Jun., 613 Washington St., Cairo, Ill., (Elected July 11, 1927.) (Age 30. Born Blackstone, Va.) 1923 B. S. in C. E. Va. Mil. Inst. 1926 M. S., Mass. Inst. Tech. *TT 4: P 4.*—June 1923 to Aug. 1924 Draftsman and Sept. to Nov. 1926 Engr., Truscon Steel Co., Norfolk, Va., detailing, designing and estimating concrete buildings. *TT 0.9: SP 0.5: P 0.4.*—Nov. 1926 to Sept. 1927 Engr., Virginia Steel Co., Richmond, Va., in charge of Drafting Room, designing, checking and estimating concrete structures. *TT 0.8: P 0.8: RC 0.8.*—Sept. 1927 to July 1931 Office and Field Engr., Portland Cement Association, Richmond, service work on design and construction of concrete structures and consulting with architects, engineers and contractors on concrete construction and design problems. *TT 3.8: P 3.8: RC 3.8.*—July 1931 to date Area Revetment Inspector, U. S. Engrs., Cairo, Ill., in charge of field engineering on 50 000 ft. of articulated concrete mattress bank protection, Mississippi River Comm., supervised placing of 150 000 cu. yd. of concrete and bank grading and construction surveys. *TT 0.8: P 0.8: RC 0.8.*—*TT 10.3: SP 0.5: P 9.8: RC 5.4.* Refers to J. A. Anderson, J. H. Cherry, T. T. Knappen, J. A. Miller, G. L. Reed, G. P. Rice.

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(10) **SOULE, JOHN EDWARD,** Jun., P. O. Box 1263, Pensacola, Fla. (Elected Oct. 21, 1924.) (Age 30. Born Chicago, Ill.) 1924 B. S. in C. E. and M. S. in C. E., Univ. of Ala. *TT 4: P 4.*—Summer 1921 Draftsman, Pensacola (Fla.) Shipbuilding Co., structural detailing and tracing, some design.—Aug. 1922 to Jan. 1923, Surveyman and Inspector on concrete bridge and highway construction, and June to July 1923 Project Engr. on general engineering and supervision of concrete highway construction, including paving and drainage structures, all for Escambia County, Fla. *TT 0.3: SP 0.3.*—July to Sept. 1923 and June to July 1924 2d Lieut. (Reserve), U. S. Army Engrs., on active duty at Ft. Bragg, N. C. —Aug. 1924 to March 1925 and March 1926 to Jan. 1929 with Virginia Bridge & Iron Co., until Dec. 1924 as Draftsman on structural detailing and tracing, and remainder of time Asst. Contr. Engr. at Birmingham, Ala., and Atlanta, Ga., on design, estimating and sales of structural steel. *TT 3.2: SP 0.2: P 3: RC 2.8: D 0.3.*—March 1925 to March 1926 Sales Engr., on design, estimating and sales of structural steel, until June 1925 for structural plant of Tampa, (Fla.) Shipbuilding & Eng. Co., then for Ingalls Iron Works Co., Tampa. *TT 1: P 1: RC 1: D 0.5.*—Jan. 1929 to date Pres. and Treas., Soule Contr. Co., Engrs. and Contrs., Pensacola, constructing bridges and drainage structures (over $100 000) for Mobile County, Ala., Escambia County, Fla., State of Florida, American Bascule Bridge

Corporation, U. S. Engr. Dept., Southern Pacific R. R., and others, also on surveys for Pensacola Harbor, for U. S. Engr. Dept., including special studies with hydraulic model, investigation of beach erosion and sand movement, surveys and location of Intracoastal Waterway and protection works; also Cons. Engr. on bridge design, boundary surveys, subdivision layout and design and recommendations for utilities, etc. *TT 3.2: P 3.2: RC 3.2: D 1.*—*TT 11.7: SP 0.5: P 11.2: RC 7: D 1.9.* Refers to G. J. Davis, Jr., E. S. Humphreys, J. W. Leroux, L. E. Lyon, A. R. Peyton, L. E. Thornton, W. E. Wheat, W. N. Woodbury.

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(14) **THOMAS, WALLACE ANDREWS,** Jun., 146 North Clay Ave., Ferguson, Mo. (Elected Nov. 11, 1929.) (Age 30. Born St. Louis, Mo.) June 1920 to Dec. 1924 Surveyman, Mississippi River Comm., on surveys, river discharge measurements and maintenance of river gauges. *TT 2.2: SP 2.2.*—April 1925 to May 1929 Civ. Engr., St. Louis Water Dept., acting as Constr. Engr. on new plant, Chf. of Party on topographic surveys (2 months) and Res. Engr. on river improvement project (1 1/3 years). *TT 4.1: P 4.1: RC 1.5.*—July to Oct. 1929 Inspector, Portland Cement Association, collaborating with contractors and engineers on construction of concrete pavements. *TT 0.3: P 0.3*—Oct. 1929 to Feb. 1930 Draftsman on plans for sewer construction in St. Ferdinand Sewer Dist., St. Louis County, until Dec. 1929 with Keck Eng. & Surveying Co., then with Edwin Hancock Eng. Co. *TT 0.2: SP 0.2.*—Feb. 1930 to date Asst. City Engr., Ferguson, Mo., on surveys and specifications for, and design and construction supervision of, streets, sewers, bridges, culverts and retaining walls. *TT 2: P 2: RC 2: D 1.*—*TT 8.8: SP 2.4: P 6.4: RC 3.5: D 1.* Refers to C. M. Daily, J. B. Dean, H. E. Frech, J. C. Pritchard, E. C. Renard, W. F. Saunders, Jr., E. E. Wall.

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(1) **THOMPSON, THORALF SANFORD,** Jun., 947 Eightieth St., Brooklyn, N. Y. (Age 31. Born Brooklyn, N. Y.) (elected March 12, 1923.) 1922 C. E., Pol. Inst. of Brooklyn. *TT 4: P 4.*—June to Sept. 1922 Jun. Asst. Engr., New York State Highway Comm., Albany, N. Y., designed highway bridge and checked design of a reinforced concrete viaduct. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—Sept. 1922 to June 1923 Instructor in Civ. Eng., Polytechnic Inst. of Brooklyn; Asst. to C. W. Hudson, Cons. Engr., in Theory of Structures, etc.; assisted in field surveying, etc. *TT 0.8: P 0.8.*—June 1923 to Sept. 1925 Estimator and Steel Designer, Geo. A. Just Co., Contr. Engrs., Long Island City, N. Y., estimated and designed structural steel for theatres, apartment houses, garages and office and Y. M. C. A. buildings. *TT 2.3: P 2.3: RC 1: D 1.*—Sept. 1925 to May 1926 Estimator with Wm. J. Wilgus, Cons. Engr., New York City, on earthwork computations and electrical overhead equipment; and checked valuation of railroad land. *TT 0.7: P 0.7.*—Aug. 1926 to July 1928 Structural Designer, until July 1927 with West Virginia Pulp & Paper Co., New York City, designed timber mill building, with long span timber roof trusses, also steel and reinforced concrete mill buildings, Aug. to Dec. 1927 with Henry R. Kent & Co., Engrs. and Constrs., Rutherford, N. J., checked design of 190-ft. steel stacks and foundations, and reinforced concrete foundations for boilers for central boiler plant of New York, New Haven & Hartford R. R., South Boston, Mass., and after Feb. 1928 with H. S. Ferguson, Cons. Engr., New York City, designed structural steel for power house and concrete foundations, also steel log sluice and stoney gates for storage dam in hydro-electric power development, High Falls, Canada. *TT 1.5: P 1.5: RC 1.2: D 1.2.*—July 1928 to date Designer, Merritt-Chapman & Scott Corporation, New York City, being Asst. on design of steel arch centers (span 110 ft.) and pier cofferdams for Washington Bridge, Providence, R. I., construction trestles for breakwaters at Manistique, Mich., and Rockaway, N. Y., steel strongbacks for laying submarine pipe (diameter 4 to 84 in., 48 to 600 ft. long), steel and timber floating derricks, and cofferdams for salvaging U. S. Govt. dredges *Manhattan* and *Raritan* (max. height dam to deck 54 ft.) *TT 3.7: P 3.7: RC 2: D 2.*—*TT 13.3: P 13.3: RC 4.5: D 4.5.* Refers to R. W. Atwater, G. A. Brinkerhoff, H. R. Codwise, H. P. Hammond, C. W. Hudson, L. A. Jenny, E. J. Squire, J. B. Whipple, J. Zimmer.

The Board of Direction will consider the applications in this list not less than thirty days after the date of its issue.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETINGS

July 4–5, 1932:

A Quarterly Meeting will be held.

SIXTY-SECOND ANNUAL CONVENTION,

July 6, 7, 8, and 9, 1932

YELLOWSTONE NATIONAL PARK

July 6, 1932:

Morning.—Technical Meeting and Annual Address of the President.

Afternoon.—Technical Meeting.

Evening.—Dinner and Entertainment.

July 7, 1932:

Morning.—Technical Division Sessions.

Afternoon.—Technical Division Sessions.

Evening.—Dinner and Entertainment.

July 8–9, 1932:

Tour of Yellowstone National Park.

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.

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